CAMANCHE EMBANKMENTS SAFETY REVIEW

VOLUME 2 OF 2
APPENDICES AND ENCLOSURES

Prepared for

EAST BAY MUNICIPAL UTILITY DISTRICT
375 Eleventh Street
Oakland, CA 94607

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TERRA / GeoPentech
a Joint Venture

Black & Veatch
AGS
Natural Resources Consulting Engineers
AP Engineering and Testing

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CLIENT:	East Bay Municipal Utility District	PROJECT:	Camanche Dam
BY: DJH	DATE: 20-Oct-08	Job No.:	162065
CHKD BY: DAS	DATE: 20-Oct-08	SHEET No.:	1 of 4
SUBJECT:	Evaluation of Erosion Potential betw een Existing Embankment Materials		

MathCAD 2001i Professional, MathSoft, Inc. c.1986-2001

Evaluation of Erosion Potential Between Materials within the Main Embankment Dam

STATEMENT: Perform an erosion potential analyses of the embankment materials.

REFERENCES:

1. United States Department of the Interior, Bureau of Reclamation 2007, Design Standards No. 13, Chapter 5 -Protective Filters.
2. Bechtel 1964, Construction Report for Camanche Dam.
3. Bechtel 1966, Construction Drawings for Camanche Dam and Dike 2 Seepage Repairs.
4. Wahler Associates 1981, Seismic Evaluation of Camanche Main Dam.
5. Wahler Associates 1983, Seismic Evaluation of Camanche Dam - Dike 2.

SCOPE:

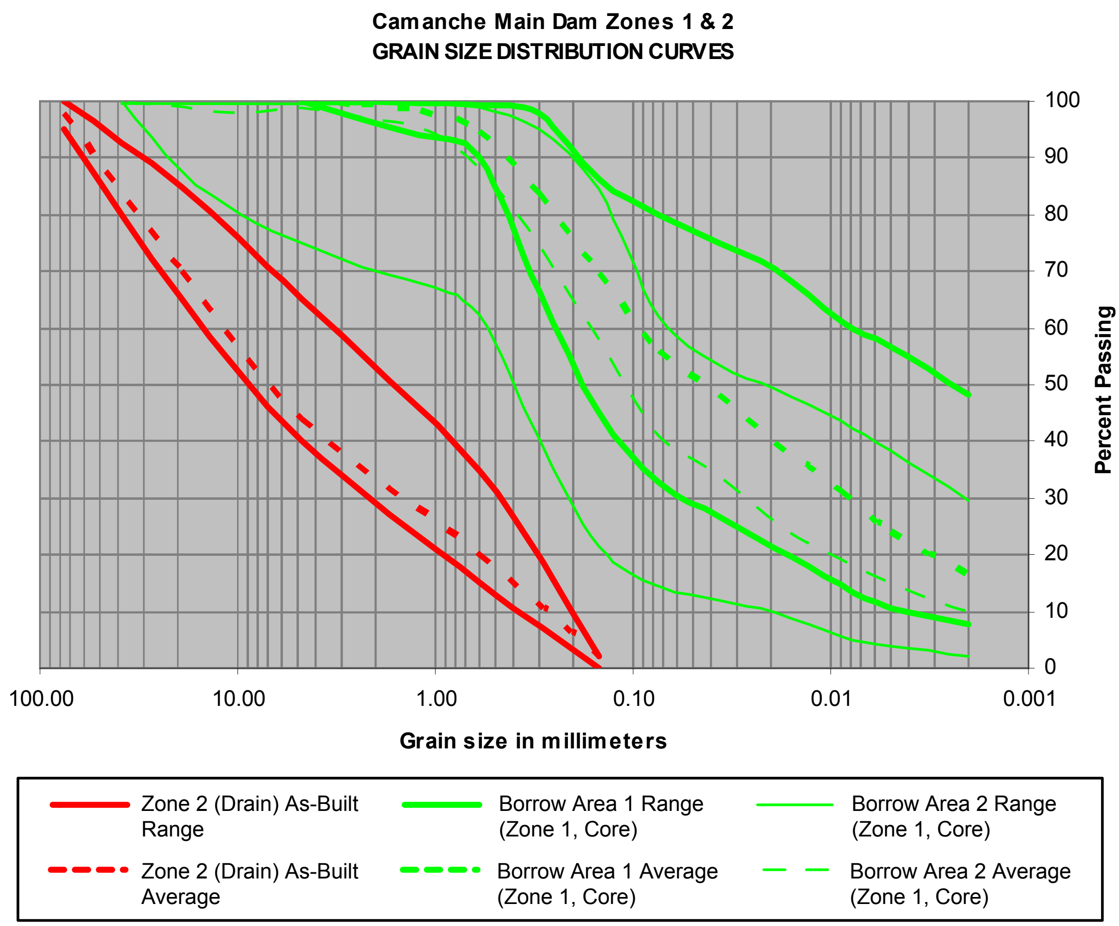
1. Determine if there is the potential for erosion to occur between Zone 1 and Zone 2 materials based on an analysis of their respective grain sizes. The analysis will follow the procedure developed for drainage blanket design.

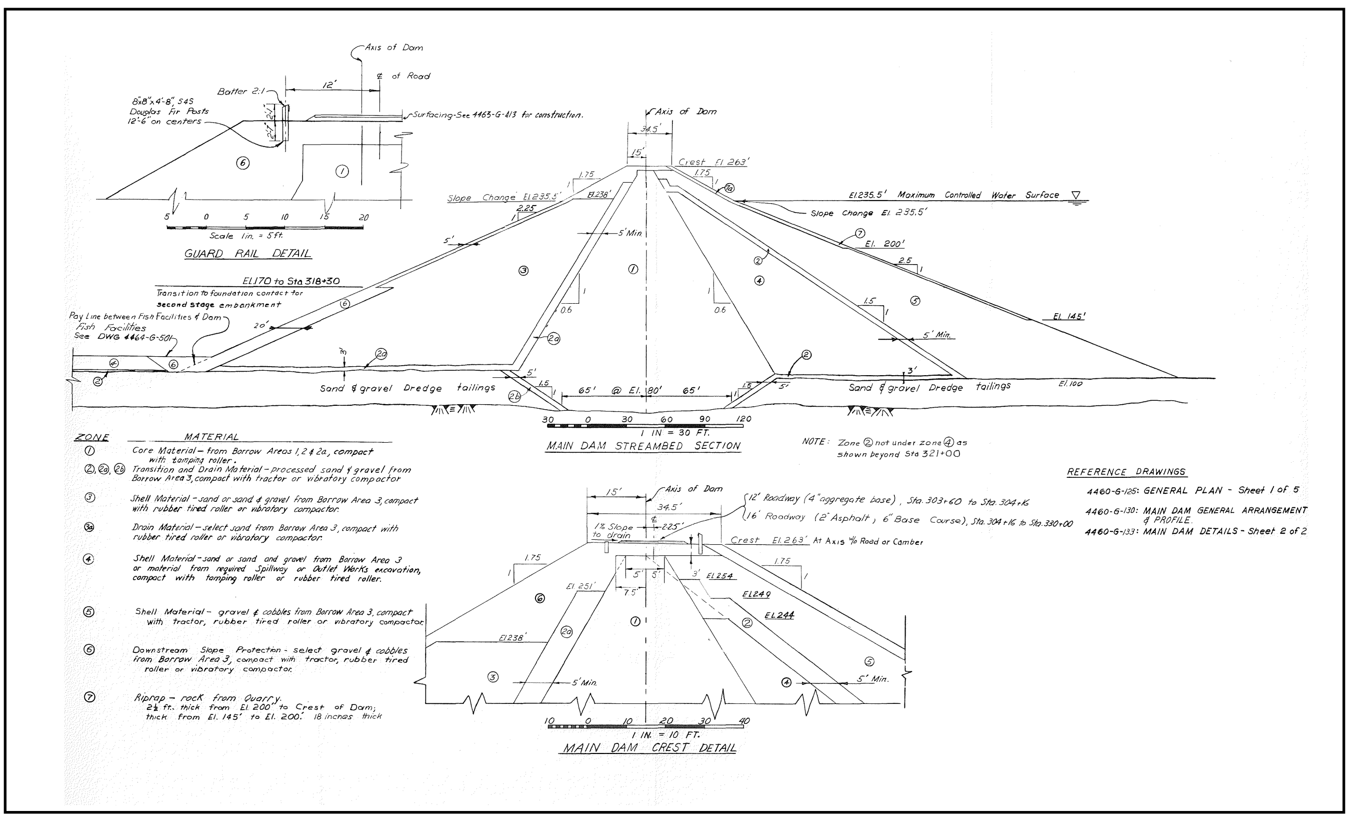
2. Soil filter and base parameters will be determined from the gradation curves presented in References 2 through 5.

Filter compatibility between the Zone 1 and Zone 2 is based on the following criteria for Type 2 base soil per USBR:

$D_{15}\,\text{Filter} \le 0.7 \cdot \text{mm}$ ***Reference 1***

Soil properties were determined from the soil gradation curves as found in References 2-5.





Main Dam Cross-Section (See Figure 2-1 of Report for Full-Size Figure)

Design Parameters

Based on the gradation curves for the embankment materials, presented above, upper and lower limit values were obtained for Zone 1 (Dam) and Zone 2 (Filter), to be used in the analyses.

Gradation Limits for the Zone 1

Lower Limits

D15DamLower :< 0.001mm

D85DamLower := 0.15mm

Upper Limits

D15DamUpper := 0.10mm

D85DamUpper := 15mm

Gradation Limits for Zone 2

Lower Limits (Fine Material)

D15FilterLower := 0.25mm

D85FilterLower := 19.0mm

Upper Limits (Coarse Material)

D15FilterUpper := 0.6mm

D85FilterUpper := 49.0mm

Determination of Filter Compatibility:

Recall: D_{15} Filter ≤ 0.7 ***Reference 1***

Lower Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p := "Lower Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q := "Lower Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of a lower limit core material being filtered by a lower limit drain material can be determined.

D15Filter2L := if(D15FilterLower ≤ 0.7·mm, p, q)

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

D15FilterLower := 0.25mm

The filter capacity of a lower limit core material being filtered by an upper limit drain material can be determined.

D15Filter4U := if(D15FilterUpper ≤ 0.7·mm, p, q)

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

D15FilterUpper := 0.6mm

Upper Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p1 := "Upper Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q1 := "Upper Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of an upper limit core material being filtered by a lower limit drain material can be determined.

D15Filter3L := if(D15FilterLower ≤ 0.7·mm, p1, q1)

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

D15FilterLower := 0.25mm

The filter capacity of an upper limit core material being filtered by an upper limit drain material can be determined.

D15Filter5U := if(D15FilterUpper ≤ 0.7mm, p1, q1)

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

D15FilterUpper := 0.6mm

Results

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

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CLIENT:	East Bay Municipal Utility District	PROJECT:	Camanche Dam
BY: DJH	DATE: 20-Oct-08	Job No.:	162065
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SUBJECT:	Evaluation of Erosion Potential betw een Existing Embankment Materials		

MathCAD 2001i Professional, MathSoft, Inc. c.1986-2001

Evaluation of Erosion Potential Between Materials within the Main Embankment Dam

STATEMENT: Perform an erosion potential analyses of the embankment materials.

REFERENCES:

1. United States Department of the Interior, Bureau of Reclamation 2007, Design Standards No. 13, Chapter 5 -Protective Filters.
2. Bechtel 1964, Construction Report for Camanche Dam.
3. Bechtel 1966, Construction Drawings for Camanche Dam and Dike 2 Seepage Repairs.
4. Wahler Associates 1981, Seismic Evaluation of Camanche Main Dam.
5. Wahler Associates 1983, Seismic Evaluation of Camanche Dam - Dike 2.

SCOPE:

1. Determine if there is the potential for erosion to occur between Zone 2 and Zone 3 materials based on an analysis of their respective grain sizes. The analysis will follow the procedure developed for drainage blanket design.

2. Soil filter and base parameters will be determined from the gradation curves presented in References 2 through 5.

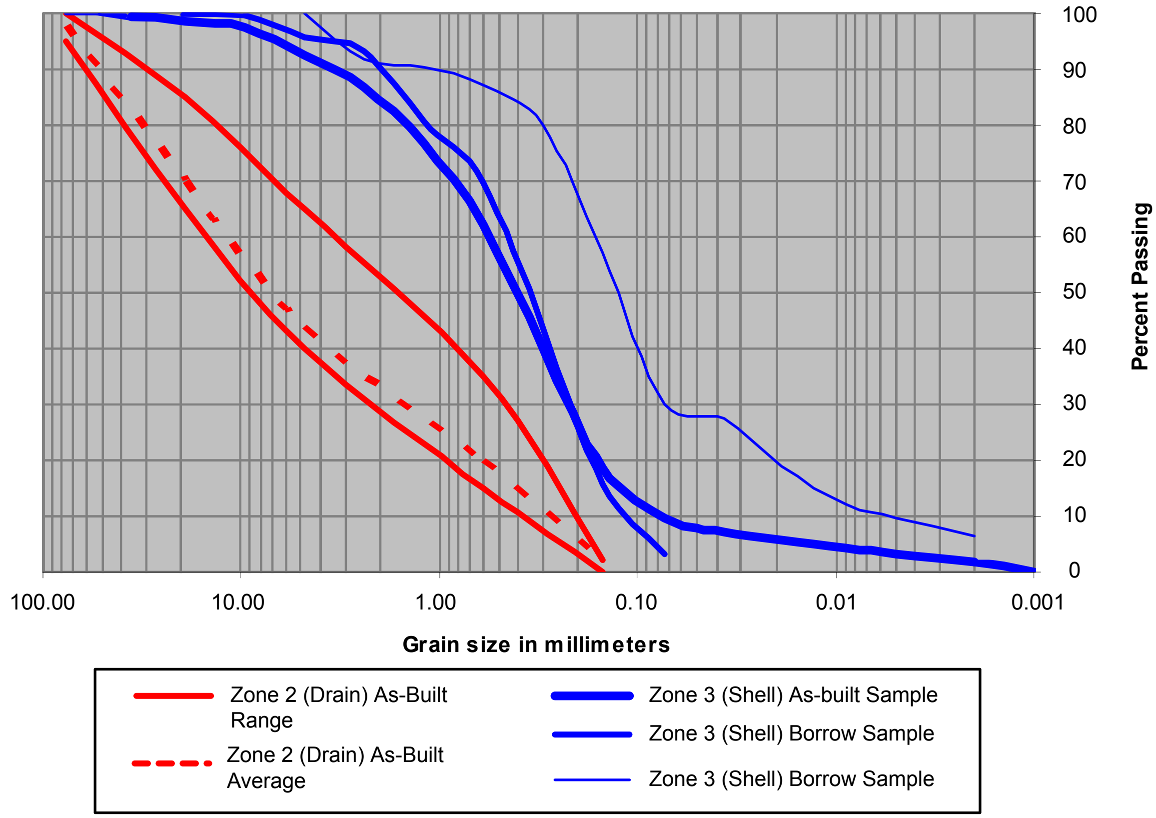
Filter compatibility between the Zone 2 and Zone 3 is based on the following criteria for Type 4 Base Soil per USBR:

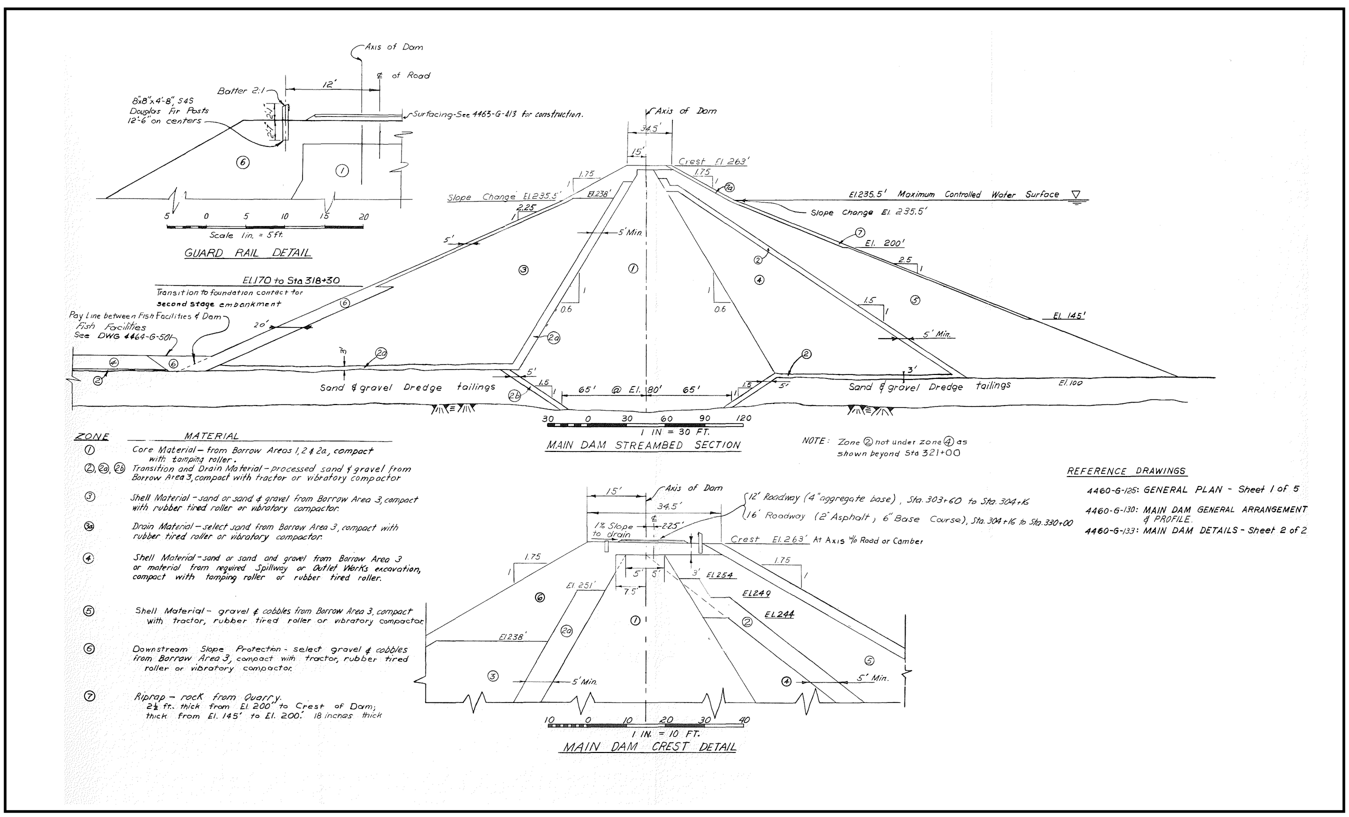
$$\frac{D_{15}\,\text{Filter}}{D_{85}\,\text{Base}} \le 4$$

Reference 1

Soil properties were determined from the soil gradation curves as found in References 2-5.

Camanche Main Dam Zones 2 & 3
GRAIN SIZE DISTRIBUTION CURVES





Main Dam Cross-Section (See Figure 2-1 of Report for Full-Size Figure)

Design Parameters

Based on the gradation curves for the embankment materials, presented above, upper and lower limit values were obtained for Zone 2 (Filter) and Zone 3 (Dam), to be used in the analyses.

Gradation Limits for the Zone 3

Lower Limits

D15DamLower := 0.013mm

D85DamLower := 0.3mm

Upper Limits

D15DamUpper := 0.15mm

D85DamUpper := 2.0mm

Gradation Limits for Zone 2

Lower Limits (Fine Material)

D15FilterLower := 0.25mm

D85FilterLower := 19.0mm

Upper Limits (Coarse Material)

D15FilterUpper := 0.6mm

D85FilterUpper := 49.0mm

Determination of Filter Compatibility:

Recall: $\dfrac{D_{15}\,\text{Filter}}{D_{85}\,\text{Base}} \le 4$ ***Reference 1***

Lower Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p := "Lower Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q := "Lower Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of a lower limit shell material being filtered by a lower limit drain material can be determined.

$$\text{D15Filter2L} := \text{if}\left(\frac{\text{D15FilterLower}}{\text{D85DamLower}} \le 4, p, q\right)$$

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

$$\frac{\text{D15FilterLower}}{\text{D85DamLower}} = 1$$

The filter capacity of a lower limit shell material being filtered by an upper limit drain material can be determined.

$$\text{D15Filter4U} := \text{if}\left(\frac{\text{D15FilterUpper}}{\text{D85DamLower}} \le 4, p, q\right)$$

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

$$\frac{\text{D15FilterUpper}}{\text{D85DamLower}} = 2$$

Upper Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p1 := "Upper Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q1 := "Upper Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of an upper limit shell material being filtered by a lower limit drain material can be determined.

$$D15Filter3L := \text{if}\left(\frac{D15FilterLower}{D85DamUpper} \le 4, p1, q1\right)$$

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

$$\frac{D15FilterLower}{D85DamUpper} = 0.125$$

The filter capacity of an upper limit shell material being filtered by an upper limit drain material can be determined.

$$D15Filter5U := \text{if}\left(\frac{D15FilterUpper}{D85DamUpper} \le 4, p1, q1\right)$$

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

$$\frac{D15FilterUpper}{D85DamUpper} = 0.3$$

Results

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

Evaluation of Erosion Potential Between Materials within the Main Embankment Dam

STATEMENT: Perform an erosion potential analyses of the embankment materials.

REFERENCES:

1. United States Department of the Interior, Bureau of Reclamation 2007, Design Standards No. 13, Chapter 5 -Protective Filters.
2. Bechtel 1964, Construction Report for Camanche Dam.
3. Bechtel 1966, Construction Drawings for Camanche Dam and Dike 2 Seepage Repairs.
4. Wahler Associates 1981, Seismic Evaluation of Camanche Main Dam.
5. Wahler Associates 1983, Seismic Evaluation of Camanche Dam - Dike 2.

SCOPE:

1. Determine if there is the potential for erosion to occur between Zone 2 and Tailings materials based on an analysis of their respective grain sizes. The analysis will follow the procedure developed for drainage blanket design.

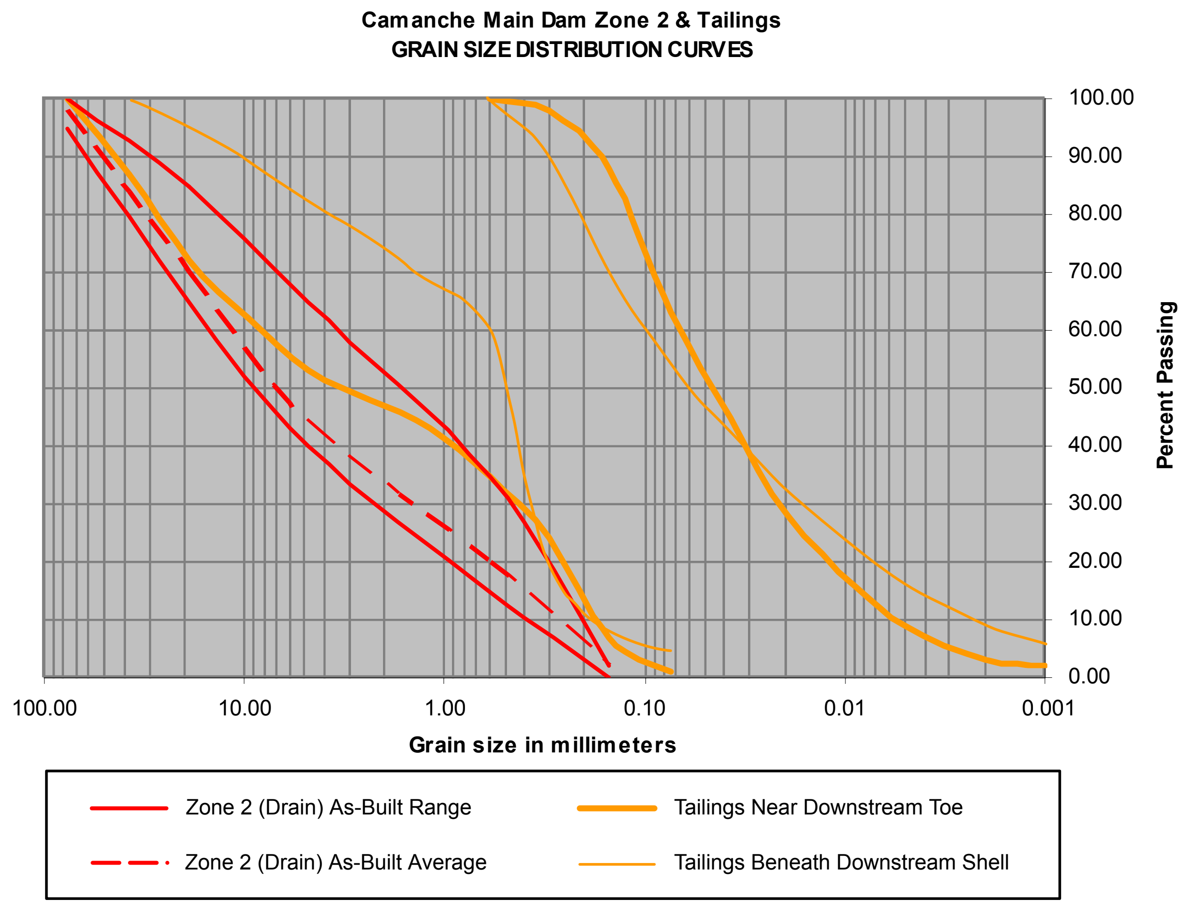
2. Soil filter and base parameters will be determined from the gradation curves presented in References 2 through 5.

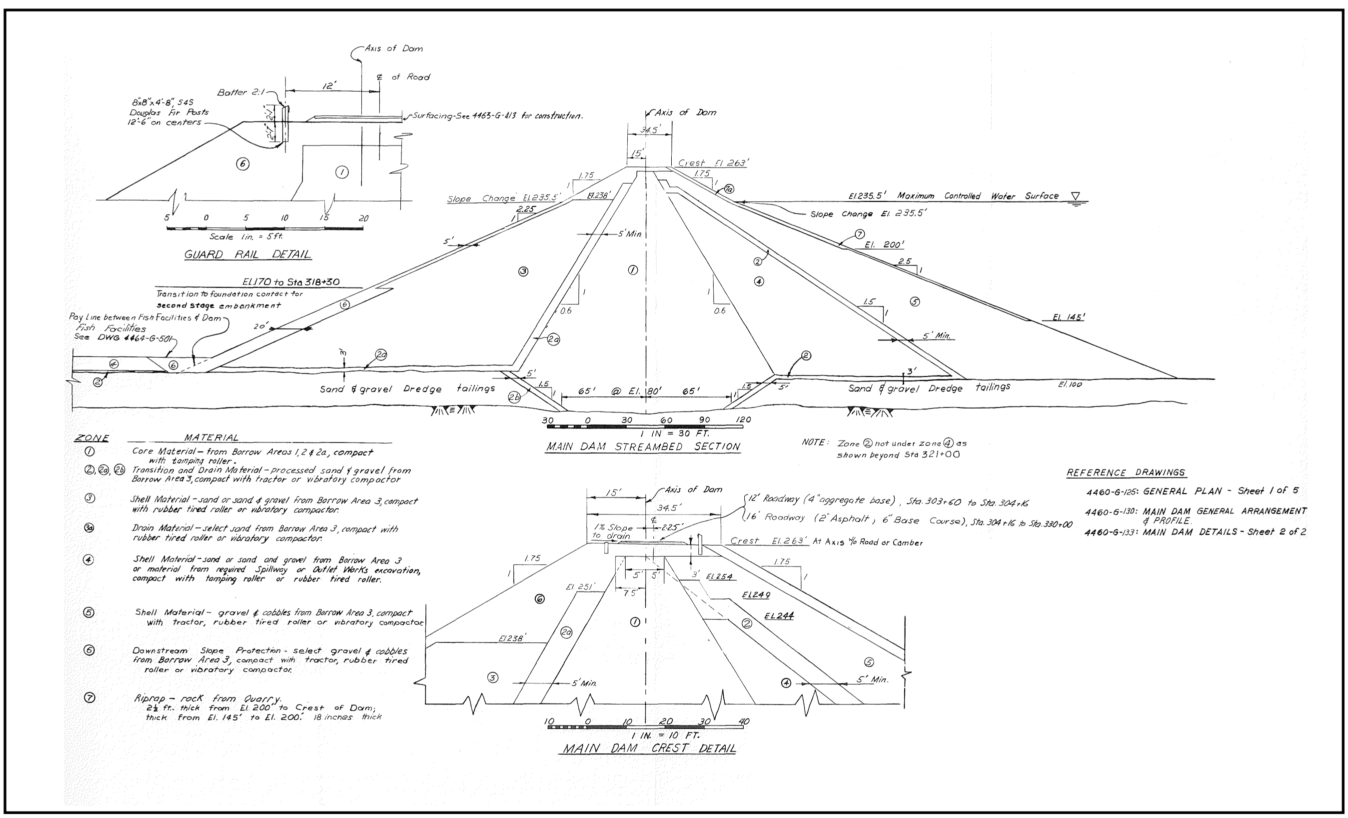
Filter compatibility between the Zone 2 and Tailings is based on the following criteria for a Type 4 Base Soil per USBR:

$$\frac{D_{15}\,\text{Filter}}{D_{85}\,\text{Base}} \le 4$$

Reference 1

Soil properties were determined from the soil gradation curves as found in References 2-5.





Main Dam Cross-Section (See Figure 2-1 of Report for Full-Size Figure)

Design Parameters

Based on the gradation curves for the embankment materials, presented above, upper and lower limit values were obtained for Tailings (Dam) and Zone 2 (Filter), to be used in the analyses.

Gradation Limits for the Tailings

Lower Limits

D15DamLower := 0.0045mm

D85DamLower := 0.16mm

Upper Limits

D15DamUpper := 0.25mm

D85DamUpper := 35.0mm

Gradation Limits for Zone 2

Lower Limits (Fine Material)

D15FilterLower := 0.25mm

D85FilterLower := 19.0mm

Upper Limits (Coarse Material)

D15FilterUpper := 0.6mm

D85FilterUpper := 49.0mm

Determination of Filter Compatibility:

Recall: $\frac{D_{15}\,\text{Filter}}{D_{85}\,\text{Base}} \leq 4$ ***Reference 1***

Lower Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:
p := "Lower Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:
q := "Lower Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of a lower limit tailings material being filtered by a lower limit drain material can be determined.

$$\text{D15Filter2L} := \text{if}\left(\frac{\text{D15FilterLower}}{\text{D85DamLower}} \leq 4, p, q\right)$$

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

$$\frac{\text{D15FilterLower}}{\text{D85DamLower}} = 2$$

The filter capacity of a lower limit tailings material being filtered by an upper limit drain material can be determined.

$$\text{D15Filter4U} := \text{if}\left(\frac{\text{D15FilterUpper}}{\text{D85DamLower}} \leq 4, p, q\right)$$

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

$$\frac{\text{D15FilterUpper}}{\text{D85DamLower}} = 4$$

Upper Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p1 := "Upper Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q1 := "Upper BoundD85 of the Filter is NOT SATISFIED"

The filter capacity of an upper limit tailings material being filtered by a lower limit drain material can be determined.

$$D15Filter3L := \text{if}\left(\frac{D15FilterLower}{D85DamUpper} \le 4, p1, q1\right)$$

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

$$\frac{D15FilterLower}{D85DamUpper} = 7.143 \times 10^{-3}$$

The filter capacity of an upper limit tailings material being filtered by an upper limit drain material can be determined.

$$D15Filter5U := \text{if}\left(\frac{D15FilterUpper}{D85DamUpper} \le 4, p1, q1\right)$$

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

$$\frac{D15FilterUpper}{D85DamUpper} = 0.017$$

Results

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

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CLIENT:	East Bay Municipal Utility District	**PROJECT:**	Camanche Dam
BY: DJH	**DATE:** 20-Oct-08	**Job No.:**	162065
CHKD BY: DAS	**DATE:** 20-Oct-08	**SHEET No.:**	1 of 4
SUBJECT:	Evaluation of Erosion Potential betw een Existing Embankment Materials in Dike No. 2		

MathCAD 2001i Professional, MathSoft, Inc. c.1986-2001

Evaluation of Erosion Potential Between Materials in Dike No. 2

STATEMENT: Perform an erosion potential analyses of the embankment materials.

REFERENCES:

1. United States Department of the Interior, Bureau of Reclamation 2007, Design Standards No. 13, Chapter 5 -Protective Filters.
2. Bechtel 1964, Construction Report for Camanche Dam.
3. Bechtel 1966, Construction Drawings for Camanche Dam and Dike 2 Seepage Repairs.
4. Wahler Associates 1981, Seismic Evaluation of Camanche Main Dam.
5. Wahler Associates 1983, Seismic Evaluation of Camanche Dam - Dike 2.

SCOPE:

1. Determine if there is the potential for erosion to occur between Zone 1 and Zone 3a materials based on an analysis of their respective grain sizes. The analysis will follow the procedure developed for drainage blanket design.

2. Soil filter and base parameters will be determined from the gradation curves presented in References 2 through 5.

Filter compatibility between the Zone 1 and Zone 3a is based on the following criteria for Type 3 base soil per USBR:

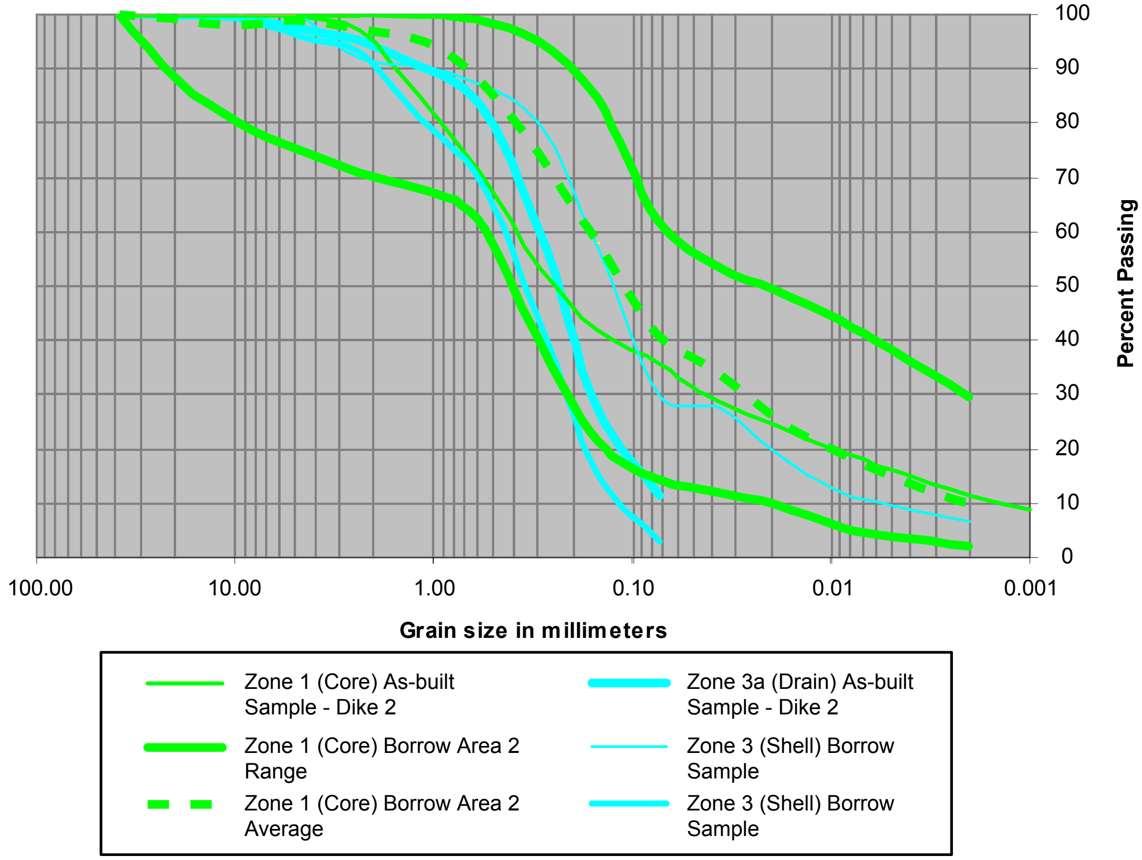
$$D_{15}\,\text{Filter} \le \left[0.7\text{mm} + (40 - A)\frac{(4D_{85}\cdot\text{Base} - 0.7\text{mm})}{25} \right]$$

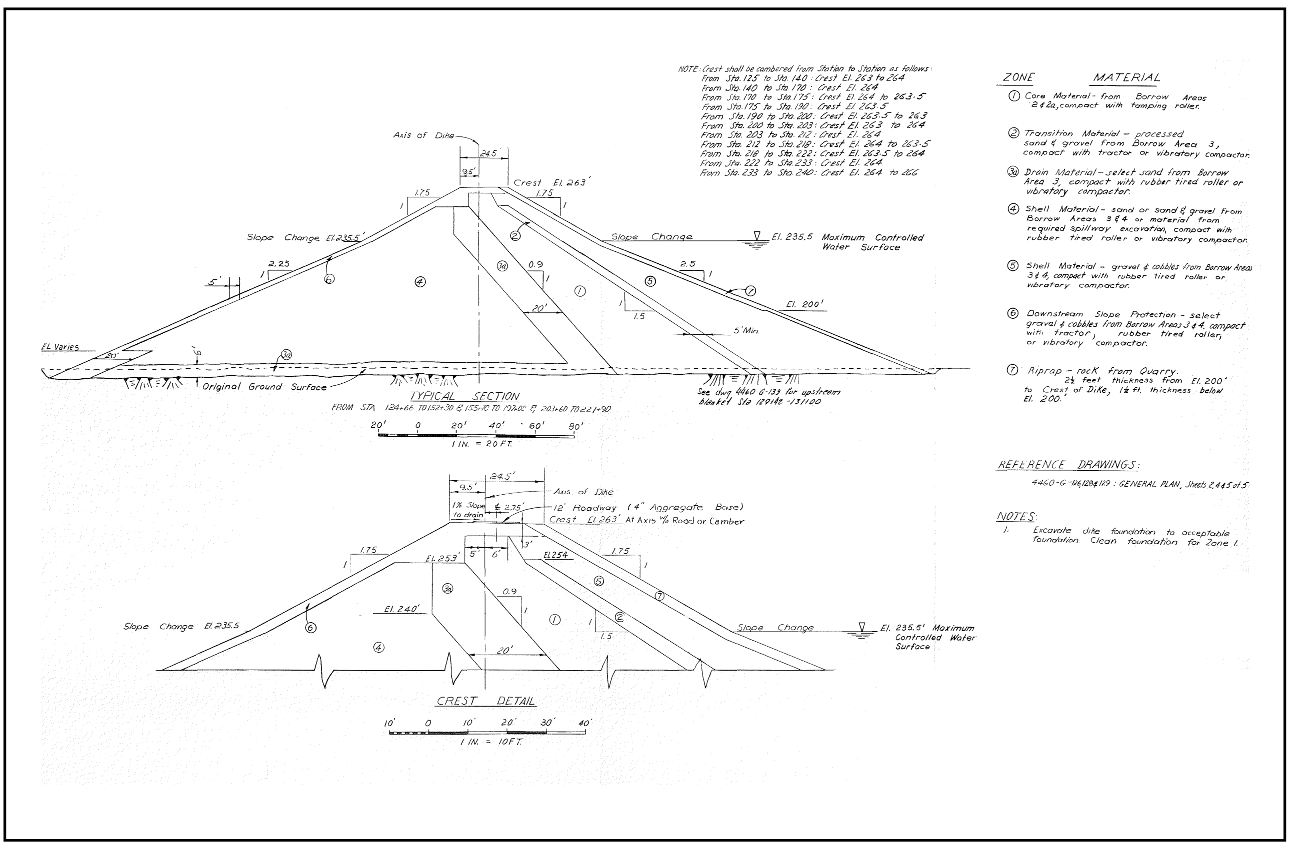
Reference 1

$A := 39$ Percent Passing #200 Sieve

Soil properties were determined from the soil gradation curves as found in References 2-5.

Camanche Dam, Dike 2 Zone 1 & 3a
GRAIN SIZE DISTRIBUTION CURVES





Dike No. 2 Typical Cross-Section (See Figure 2-2 of Report for Full-Size Figure)

Design Parameters

Based on the gradation curves for the embankment materials, presented above, upper and lower limit values were obtained for Zone 1 (Dam) andZone 3a (Filter), to be used in the analyses.

Gradation Limits for the Zone 1

Lower Limits

D15DamLower := 0.001mm

D85DamLower := 0.15mm

Upper Limits

D15DamUpper := 0.003mm

D85DamUpper := 1.2mm

Gradation Limits for Zone 3a

Lower Limits (Fine Material)

D15FilterLower := 0.015mm

D85FilterLower := 0.4mm

Upper Limits (Coarse Material)

D15FilterUpper := 0.15mm

D85FilterUpper := 1.5mm

Determination of Filter Compatibility:

Recall: $D_{15}\,\text{Filter} \le \left[0.7\text{mm} + (40 - A)\dfrac{(4D_{85}\cdot\text{Base} - 0.7\text{mm})}{25}\right]$ ***Reference 1***

Lower Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p := "Lower Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q := "Lower Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of a lower limit core material being filtered by a lower limit drain material can be determined.

$$\text{D15Filter2L} := \text{if}\left[\text{D15FilterLower} \le \left[0.7\text{mm} + (40 - 39)\frac{(4\text{D85DamLower} - 0.7\text{mm})}{25}\right], p, q\right]$$

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

$$\text{Criteria}_{\text{Adjusted}} := 0.7\cdot\text{mm} + (40 - 39)\cdot\frac{(4\cdot\text{D85DamLower} - 0.7\cdot\text{mm})}{25}$$

$\text{Criteria}_{\text{Adjusted}} = 0.7\,\text{mm}$ D15FilterLower := 0.015mm

The filter capacity of a lower limit core material being filtered by an upper limit drain material can be determined.

$$\text{D15Filter4U} := \text{if}\left[\text{D15FilterUpper} \le 0.7\cdot\text{mm} + (40 - 39)\cdot\frac{(4\cdot\text{D85DamLower} - 0.7\cdot\text{mm})}{25}, p, q\right]$$

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

$\text{Criteria}_{\text{Adjusted}} = 0.7\,\text{mm}$ D15FilterUpper := 0.15mm

Upper Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p1 := "Upper Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q1 := "Upper Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of an upper limit core material being filtered by a lower limit drain material can be determined.

$$D15Filter3L := \text{if}\left(D15FilterLower \le 0.7\cdot mm + (40-39)\cdot\frac{(4\cdot D85DamUpper - 0.7\cdot mm)}{25}, p1, q1\right)$$

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

$$Criteria_{Adjusted2} := 0.7\cdot mm + (40-39)\cdot\frac{(4\cdot D85DamUpper - 0.7\cdot mm)}{25}$$

$Criteria_{Adjusted2} = 0.86\,mm$ D15FilterLower := 0.015mm

The filter capacity of an upper limit core material being filtered by an upper limit drain material can be determined.

$$D15Filter5U := \text{if}\left(D15FilterUpper \le 0.7\cdot mm + (40-39)\cdot\frac{(4\cdot D85DamUpper - 0.7\cdot mm)}{25}, p1, q1\right)$$

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

$Criteria_{Adjusted2} = 0.86\,mm$ D15FilterUpper := 0.15mm

Results

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

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CLIENT:	East Bay Municipal Utility District	**PROJECT:**	Camanche Dam
BY: DJH	**DATE:** 20-Oct-08	**Job No.:**	162065
CHKD BY: DAS	**DATE:** 20-Oct-08	**SHEET No.:**	1 of 4

SUBJECT: Evaluation of Erosion Potential between Existing Embankment Materials in Dike No. 2

MathCAD 2001i Professional, MathSoft, Inc. c.1986-2001

Evaluation of Erosion Potential Between Materials in Dike No. 2

STATEMENT: Perform an erosion potential analyses of the embankment materials.

REFERENCES:

1. United States Department of the Interior, Bureau of Reclamation 2007, Design Standards No. 13, Chapter 5 -Protective Filters.
2. Bechtel 1964, Construction Report for Camanche Dam.
3. Bechtel 1966, Construction Drawings for Camanche Dam and Dike 2 Seepage Repairs.
4. Wahler Associates 1981, Seismic Evaluation of Camanche Main Dam.
5. Wahler Associates 1983, Seismic Evaluation of Camanche Dam - Dike 2.

SCOPE:

1. Determine if there is the potential for erosion to occur between Zone 1 and Alluvium materials based on an analysis of their respective grain sizes. The analysis will follow the procedure developed for drainage blanket design.

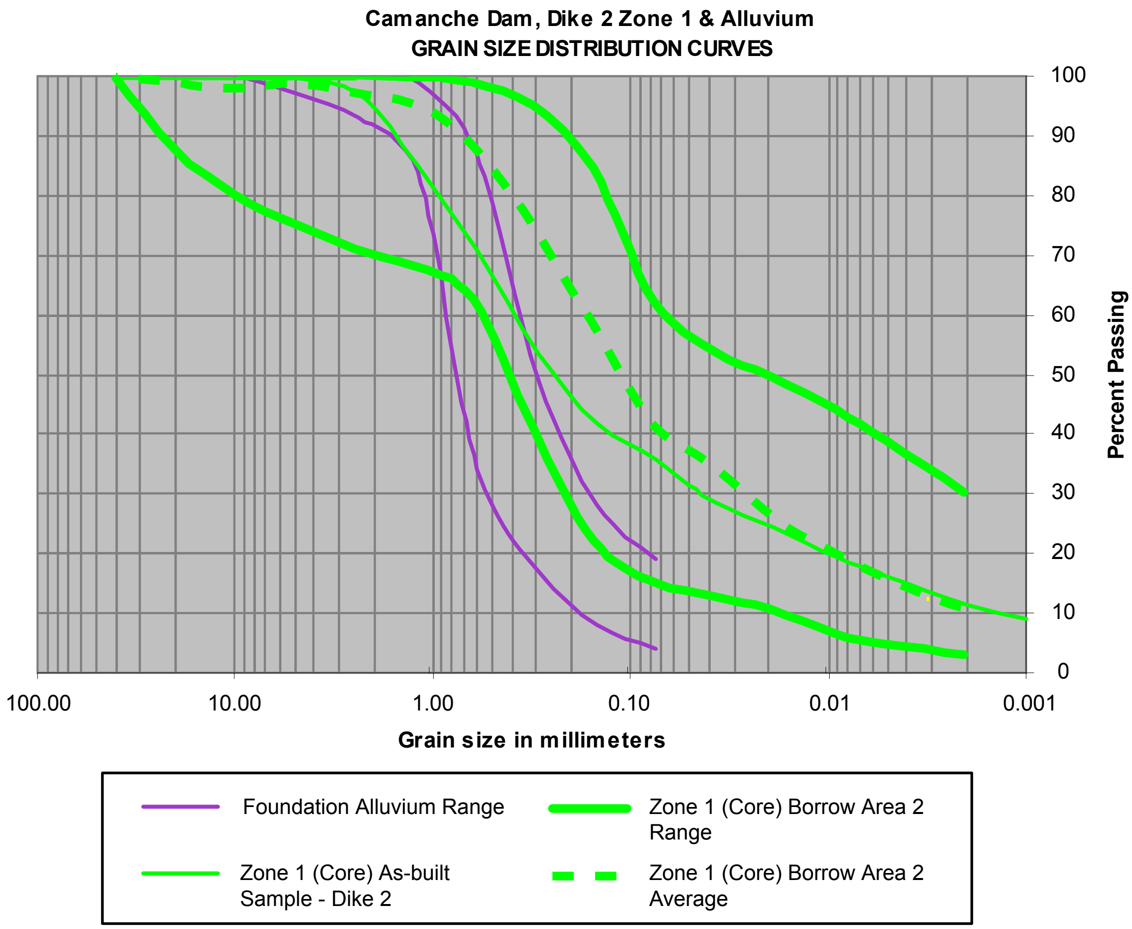
2. Soil filter and base parameters will be determined from the gradation curves presented in References 2 through 5.

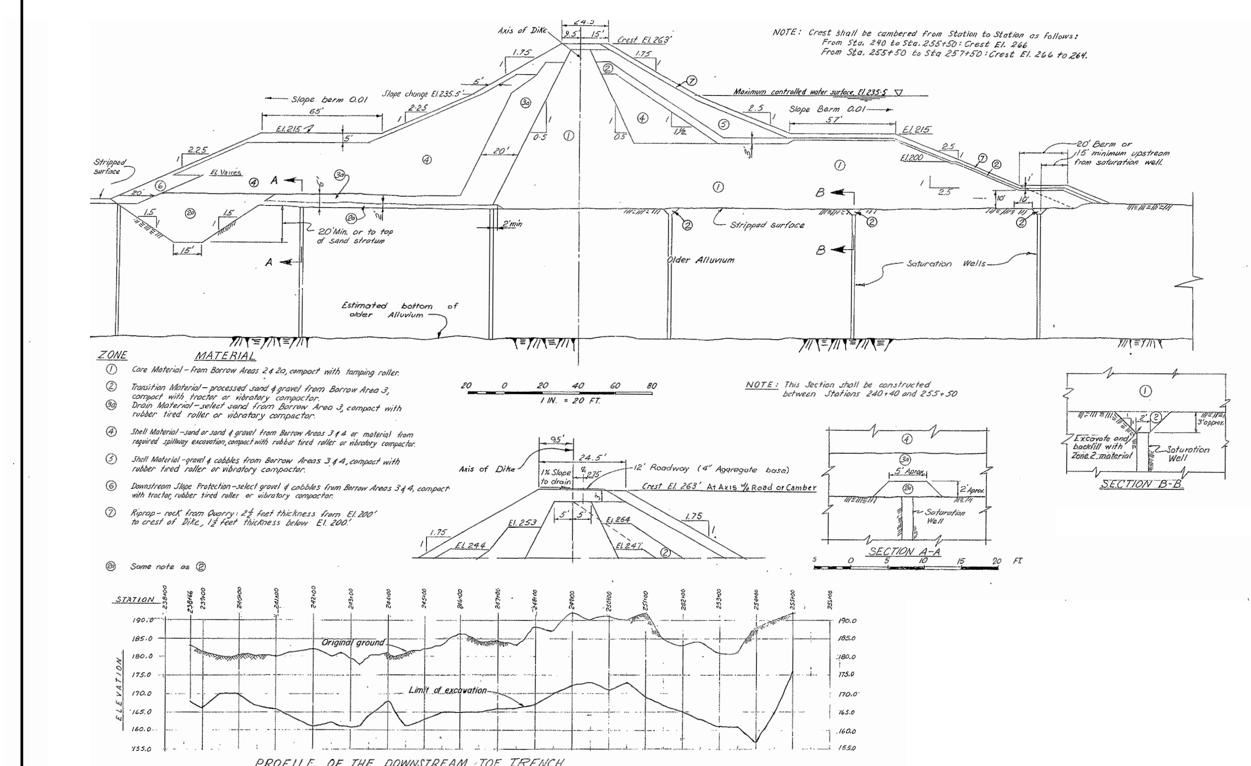
<u>Filter compatibility between the Zone 1 and Alluvium is based on the following criteria for Type 3 base soil per USBR:</u>

$$D_{15}\,\text{Filter} \le \left[0.7\text{mm} + (40 - A)\frac{(4D_{85}\cdot\text{Base} - 0.7\text{mm})}{25}\right]$$ ***Reference 1***

$A := 39$ Percent Passing #200 Sieve

Soil properties were determined from the soil gradation curves as found in References 2-5.





Dike No. 2 Deep Cross-Section (See Figure 2-3 of Report for Full-Size Figure)

Design Parameters

Based on the gradation curves for the embankment materials, presented above, upper and lower limit values were obtained for Zone 1 (Dam) and Alluvium (Filter), to be used in the analyses.

Gradation Limits for the Zone 1

Lower Limits

$D15DamLower := 0.001mm$

$D85DamLower := 0.15mm$

Upper Limits

$D15DamUpper := 0.003mm$

$D85DamUpper := 1.2mm$

Gradation Limits for Alluvium

Lower Limits (Fine Material)

$D15FilterLower := 0.05mm$

$D85FilterLower := 0.55mm$

Upper Limits (Coarse Material)

$D15FilterUpper := 0.25mm$

$D85FilterUpper := 1.2mm$

Determination of Filter Compatibility:

Recall: $D_{15}Filter \le \left(0.7mm + (40 - A)\frac{(4D_{85}\cdot Base - 0.7mm)}{25}\right)$ ***Reference 1***

Lower Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p := "Lower Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q := "Lower Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of a lower limit core material being filtered by a lower limit foundation material can be determined.

$$D15Filter2L := if\left(D15FilterLower \le 0.7\cdot mm + (40 - 39)\cdot\frac{(4\cdot D85DamLower - 0.7\cdot mm)}{25}, p, q\right)$$

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

$$Criteria_{Adjusted} := 0.7\cdot mm + (40 - 39)\cdot\frac{(4\cdot D85DamLower - 0.7\cdot mm)}{25}$$

$Criteria_{Adjusted} = 0.7\,mm$ $D15FilterLower := 0.05mm$

The filter capacity of a lower limit core material being filtered by an upper limit foundation material can be determined.

$$D15Filter4U := if\left(\frac{D15FilterUpper}{D85DamLower} \le 4, p, q\right)$$

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

$Criteria_{Adjusted} = 0.7\,mm$ $D15FilterUpper := 0.25mm$

Upper Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p1 := "Upper Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q1 := "Upper Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of an upper limit core material being filtered by a lower limit foundation material can be determined.

$$\text{D15Filter3L} := \text{if}\left(\text{D15FilterLower} \le 0.7\cdot\text{mm} + (40 - 39)\cdot\frac{(4\cdot\text{D85DamUpper} - 0.7\cdot\text{mm})}{25}, \text{p1}, \text{q1}\right)$$

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

$$\text{Criteria}_{\text{Adjusted2}} := 0.7\cdot\text{mm} + (40 - 39)\cdot\frac{(4\cdot\text{D85DamUpper} - 0.7\cdot\text{mm})}{25}$$

$\text{Criteria}_{\text{Adjusted2}} = 0.86\,\text{mm}$ D15FilterLower := 0.05mm

The filter capacity of an upper limit core material being filtered by an upper limit foundation material can be determined.

$$\text{D15Filter5U} := \text{if}\left(\frac{\text{D15FilterUpper}}{\text{D85DamUpper}} \le 4, \text{p1}, \text{q1}\right)$$

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

$\text{Criteria}_{\text{Adjusted2}} = 0.86\,\text{mm}$ D15FilterUpper := 0.25mm

Results

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

Evaluation of Erosion Potential Between Materials in Dike No. 2

STATEMENT: Perform an erosion potential analyses of the embankment materials.

REFERENCES:

1. United States Department of the Interior, Bureau of Reclamation 2007, Design Standards No. 13, Chapter 5 -Protective Filters.
2. Bechtel 1964, Construction Report for Camanche Dam.
3. Bechtel 1966, Construction Drawings for Camanche Dam and Dike 2 Seepage Repairs.
4. Wahler Associates 1981, Seismic Evaluation of Camanche Main Dam.
5. Wahler Associates 1983, Seismic Evaluation of Camanche Dam - Dike 2.

SCOPE:

1. Determine if there is the potential for erosion to occur between Zone 3a and Zone 4 materials based on an analysis of their respective grain sizes. The analysis will follow the procedure developed for drainage blanket design.

2. Soil filter and base parameters will be determined from the gradation curves presented in References 2 through 5.

Filter compatibility between the Zone 3a and Zone 4 is based on the following criteria for Type 3 base soil per USBR:

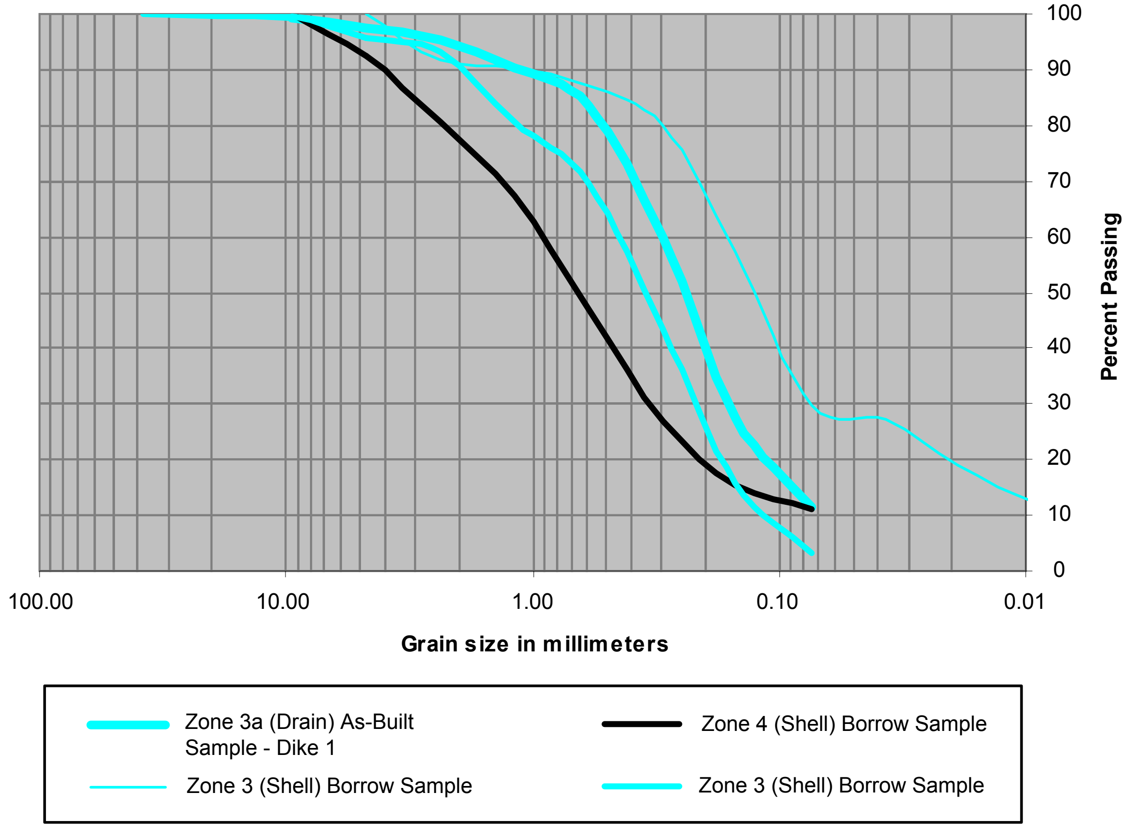
$$D_{15}\,\text{Filter} \le \left[0.7\text{mm} + (40 - A)\frac{(4D_{85}\cdot\text{Base} - 0.7\text{mm})}{25}\right]$$

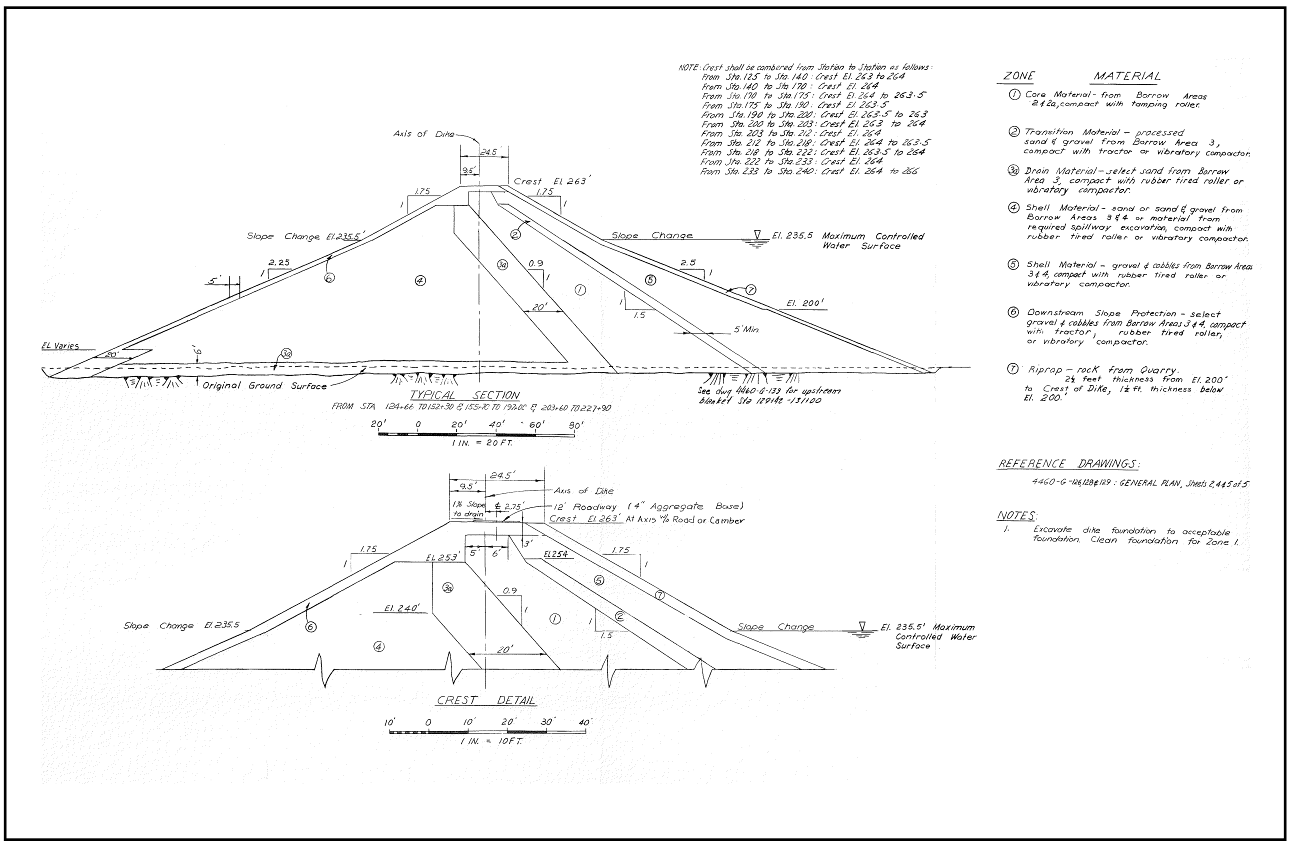
Reference 1

$A := 30$ Percent Passing #200 Sieve

Soil properties were determined from the soil gradation curves as found in References 2-5.

Camanche Dam, Dike 2 Zone 3a & 4
GRAIN SIZE DISTRIBUTION CURVES





Dike No. 2 Typical Cross-Section (See Figure 2-2 of Report for Full-Size Figure)

Design Parameters

Based on the gradation curves for the embankment materials, presented above, upper and lower limit values were obtained for Zone 4 (Dam) and Zone 3a (Filter), to be used in the analyses.

Gradation Limits for the Zone 4

Lower Limits

D15DamLower := 0.15mm

D85DamLower := 3.0mm

Upper Limits

D15DamUpper := 0.15mm

D85DamUpper := 3.0mm

Gradation Limits for Zone 3a

Lower Limits (Fine Material)

D15FilterLower := 0.015mm

D85FilterLower := 0.4mm

Upper Limits (Coarse Material)

D15FilterUpper := 0.15mm

D85FilterUpper := 1.5mm

Determination of Filter Compatibility:

Recall: $$D_{15}\,\text{Filter} \le \left(0.7\text{mm} + (40 - A)\frac{(4D_{85}\cdot\text{Base} - 0.7\text{mm})}{25}\right)$$ ***Reference 1***

Lower Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p := "Lower Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q := "Lower Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of a lower limit shell material being filtered by a lower limit drain material can be determined.

$$\text{D15Filter2L} := \text{if}\left(\text{D15FilterLower} \le 0.7\cdot\text{mm} + (40 - 30)\cdot\frac{(4\cdot\text{D85DamLower} - 0.7\cdot\text{mm})}{25}, p, q\right)$$

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

$$\text{Criteria}_{\text{Adjusted}} := 0.7\cdot\text{mm} + (40 - 30)\cdot\frac{(4\cdot\text{D85DamLower} - 0.7\cdot\text{mm})}{25}$$

$\text{Criteria}_{\text{Adjusted}} = 5.22\ \text{mm}$ D15FilterLower := 0.015mm

The filter capacity of a lower limit shell material being filtered by an upper limit drain material can be determined.

$$\text{D15Filter4U} := \text{if}\left(\text{D15FilterUpper} \le 0.7\cdot\text{mm} + (40 - 30)\cdot\frac{(4\cdot\text{D85DamLower} - 0.7\cdot\text{mm})}{25}, p, q\right)$$

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

$\text{Criteria}_{\text{Adjusted}} = 5.22\ \text{mm}$ D15FilterUpper := 0.15mm

Upper Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p1 := "Upper Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q1 := "Upper Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of an upper limit shell material being filtered by a lower limit drain material can be determined.

$$D15Filter3L := \text{if}\left(D15FilterLower \le 0.7\cdot mm + (40-30)\cdot\frac{(4\cdot D85DamUpper - 0.7\cdot mm)}{25}, p1, q1\right)$$

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

$$Criteria_{Adjusted2} := 0.7\cdot mm + (40-30)\cdot\frac{(4\cdot D85DamUpper - 0.7\cdot mm)}{25}$$

$Criteria_{Adjusted2} = 5.22\,mm$ D15FilterLower := 0.015mm

The filter capacity of an upper limit shell material being filtered by an upper limit drain material can be determined.

$$D15Filter5U := \text{if}\left(D15FilterUpper \le 0.7\cdot mm + (40-30)\cdot\frac{(4\cdot D85DamUpper - 0.7\cdot mm)}{25}, p1, q1\right)$$

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

$Criteria_{Adjusted2} = 5.22\,mm$ D15FilterUpper := 0.15mm

Results

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

CLIENT:	East Bay Municipal Utility District			**PROJECT:**	Camanche Dam
BY:	DJH	**DATE:**	20-Oct-08	**Job No.:**	162065
CHKD BY:	DAS	**DATE:**	20-Oct-08	**SHEET No.:**	1 of 4
SUBJECT:	Evaluation of Erosion Potential betw een Existing Embankment Materials in Dike No. 2				

MathCAD 2001 Professional, MathSoft, Inc. c.1986-2001

Evaluation of Erosion Potential Between Materials in Dike No. 2

STATEMENT: Perform an erosion potential analyses of the embankment materials.

REFERENCES:

1. United States Department of the Interior, Bureau of Reclamation 2007, Design Standards No. 13, Chapter 5 -Protective Filters.
2. Bechtel 1964, Construction Report for Camanche Dam.
3. Bechtel 1966, Construction Drawings for Camanche Dam and Dike 2 Seepage Repairs.
4. Wahler Associates 1981, Seismic Evaluation of Camanche Main Dam.
5. Wahler Associates 1983, Seismic Evaluation of Camanche Dam - Dike 2.

SCOPE:

1. Determine if there is the potential for erosion to occur between Zone 3a and Alluvium materials based on an analysis of their respective grain sizes. The analysis will follow the procedure developed for drainage blanket design.
2. Soil filter and base parameters will be determined from the gradation curves presented in References 2 through 5.

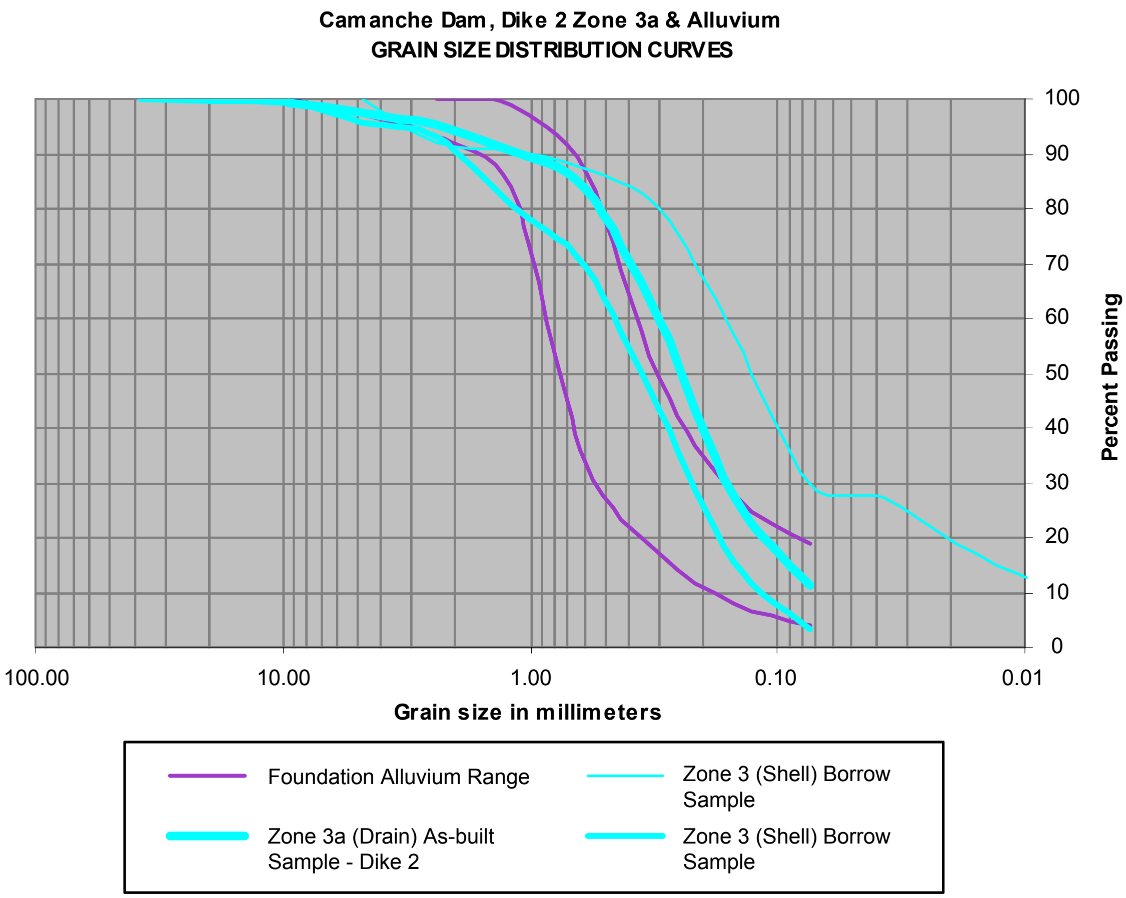
Filter compatibility between the Zone 3a and Alluvium is based on the following criteria for Type 3 base soil per USBR:

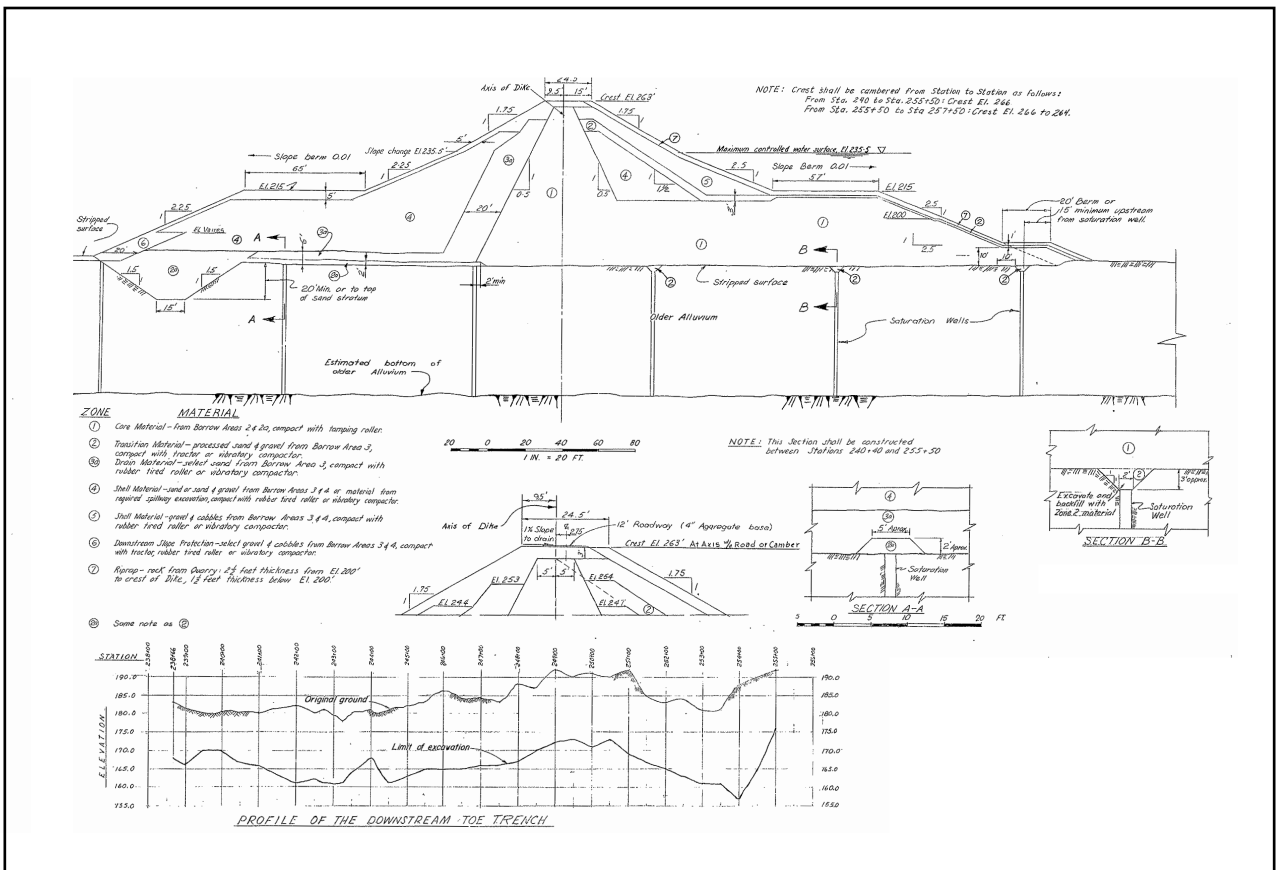
$$D_{15}\,\text{Filter} \le \left[0.7\text{mm} + (40 - A)\frac{(4D_{85}\cdot\text{Base} - 0.7\text{mm})}{25}\right]$$

Reference 1

$A := 30$ Percent Passing #200 Sieve

Soil properties were determined from the soil gradation curves as found in References 2-5.





Dike No. 2 Deep Cross-Section (See Figure 2-3 of Report for Full-Size Figure)

Design Parameters

Based on the gradation curves for the embankment materials, presented above, upper and lower limit values were obtained for Zone 3a (Dam) and Alluvium (Filter), to be used in the analyses.

Gradation Limits for the Zone 3a

Lower Limits	**Upper Limits**
D15DamLower := 0.015mm	D15DamUpper := 0.15mm
D85DamLower := 0.4mm	D85DamUpper := 1.5mm

Gradation Limits for Alluvium

Lower Limits (Fine Material)	**Upper Limits (Coarse Material)**
D15FilterLower := 0.05mm	D15FilterUpper := 0.25mm
D85FilterLower := 0.55mm	D85FilterUpper := 1.2mm

Determination of Filter Compatibility:

Recall:

$$D_{15}\,\text{Filter} \le \left[0.7\text{mm} + (40 - A)\frac{\left(4D_{85}\cdot\text{Base} - 0.7\text{mm}\right)}{25}\right]$$

Reference 1

Lower Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p := "Lower Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q := "Lower Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of a lower limit drain material being filtered by a lower limit foundation material can be determined.

$$\text{D15Filter2L} := \text{if}\left[\text{D15FilterLower} \le 0.7\cdot\text{mm} + (40 - 30)\cdot\frac{(4\cdot\text{D85DamLower} - 0.7\cdot\text{mm})}{25}, p, q\right]$$

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

$$\text{Criteria}_{\text{Adjusted}} := 0.7\cdot\text{mm} + (40 - 30)\cdot\frac{(4\cdot\text{D85DamLower} - 0.7\cdot\text{mm})}{25}$$

$\text{Criteria}_{\text{Adjusted}} = 1.06\,\text{mm}$ D15FilterLower := 0.05mm

The filter capacity of a lower limit drain material being filtered by an upper limit foundation material can be determined.

$$\text{D15Filter4U} := \text{if}\left[\text{D15FilterUpper} \le 0.7\cdot\text{mm} + (40 - 30)\cdot\frac{(4\cdot\text{D85DamLower} - 0.7\cdot\text{mm})}{25}, p, q\right]$$

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

$\text{Criteria}_{\text{Adjusted}} = 1.06\,\text{mm}$ D15FilterUpper := 0.25mm

Upper Bound

Based on the specific condition addressed, if a satisfactory result is obtained, the following will be true:

p1 := "Upper Bound of D85 of the Filter is SATISFIED"

Based on the specific condition addressed, if an unsatisfactory result is obtained, the following will be true:

q1 := "Upper Bound of D85 of the Filter is NOT SATISFIED"

The filter capacity of an upper limit drain material being filtered by a lower limit foundation material can be determined.

$$D15Filter3L := if\left(D15FilterLower \le 0.7\cdot mm + (40 - 30)\cdot\frac{(4\cdot D85DamUpper - 0.7\cdot mm)}{25}, p1, q1\right)$$

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

$$Criteria_{Adjusted} := 0.7\cdot mm + (40 - 30)\cdot\frac{(4\cdot D85DamUpper - 0.7\cdot mm)}{25}$$

$Criteria_{Adjusted} = 2.82\ mm$

D15FilterLower := 0.05mm

The filter capacity of an upper limit drain material being filtered by an upper limit foundation material can be determined.

$$D15Filter5U := if\left(D15FilterUpper \le 0.7\cdot mm + (40 - 30)\cdot\frac{(4\cdot D85DamUpper - 0.7\cdot mm)}{25}, p1, q1\right)$$

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

$Criteria_{Adjusted} = 2.82\ mm$

D15FilterUpper := 0.25mm

Results

D15Filter2L = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter4U = "Lower Bound of D85 of the Filter is SATISFIED"

D15Filter3L = "Upper Bound of D85 of the Filter is SATISFIED"

D15Filter5U = "Upper Bound of D85 of the Filter is SATISFIED"

APPENDIX B
UNIVERSITY OF NEW SOUTH WALES (UNSW) METHOD
ANNUALIZED FAILURE POTENTIAL

BLACK & VEATCH
Building a world of difference.

CLIENT:	East Bay Municipal Utility District	**PROJECT:**	Camanche Dam
BY: DJH	**DATE:** 3-Jul-08	**Job No.:**	162065
CHKD BY: DAS	**DATE:** 20-Oct-08	**SHEET No.:**	1 of 6

SUBJECT: Annualized Probability of Failure for Main Embankment Dam

MathCAD 2001i Professional, MathSoft, Inc. c.1986-2001

Evaluation of Annual Potential of Failure for the Main Embankment Dam

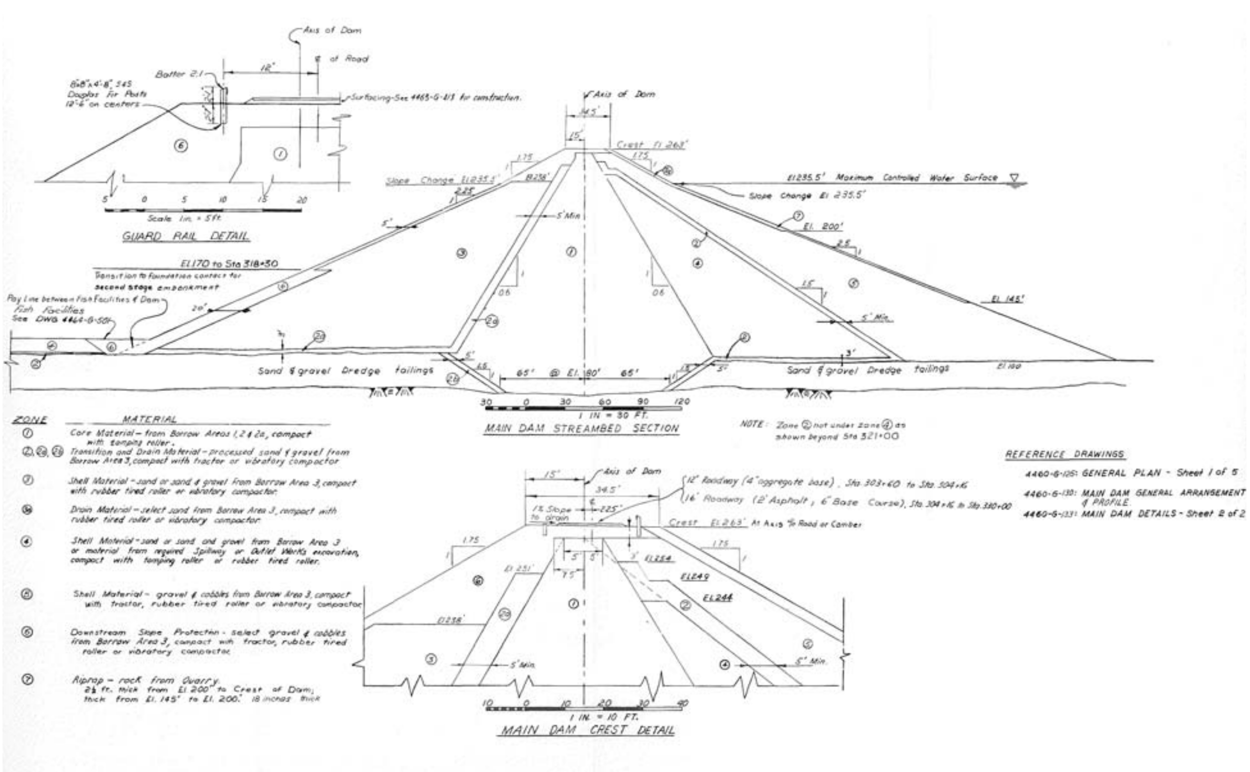
STATEMENT: Assess the annual probability of failure for the Main Embankment Dam due to piping.

REFERENCES:

1. M. Foster, R. Fell, and M. Spannagle, A Method for Assessing the Relative Likelihood of Failure of Embankment Dams by Piping, Canadian Geotechnical Journal, 2000, Vol. 37 No.5.

SCOPE:

1. Quantify the necessary components of the main dam and their current condition.
2. Apply weighting factors for specific factors associated with failure of the embankment dam due to piping.



Main Dam Cross-Section (See Figure 2-1 of Report for Full-Size Figure)

1. Determine the average annual probabilities of failure for each of three primary modes of failure:

i - piping through embankment
ii - piping through foundation
iii - piping from the embankment into the foundation

Dam was constructed in 1964 (older than 5 years).

Table 1. Average historic frequency of failure of embankment dams by mode of failure and dam zoning. Foster et al. (2000)

	Embankment			Foundation			Embankment into foundation		
		Average annual P_e (10^{-6})			Average annual P_f (10^{-6})			Average annual P_{ef} (10^{-6})	
Zoning category	Average P_{Te} (10^{-3})	First 5 years operation	After 5 years operation	Average P_{Tf} (10^{-3})	First 5 years operation	After 5 years operation	Average P_{Tef} (10^{-3})	First 5 years operation	After 5 years operation
Homogeneous earthfill	16	2080	190	1.7	255	19	0.18	19	4
Earthfill with filter	1.5	190	37	1.7	255	19	0.18	19	4
Earthfill with rock toe	8.9	1160	160	1.7	255	19	0.18	19	4
Zoned earthfill	1.2	160	25	1.7	255	19	0.18	19	4
Zoned earth and rockfill	1.2	150	24	1.7	255	19	0.18	19	4
Central core earth and rockfill	(<1)	(<140)	(<34)	1.7	255	19	0.18	19	4
Concrete face earthfill	5.3	690	75	1.7	255	19	0.18	19	4
Concrete face rockfill	(<1)	(<130)	(<17)	1.7	255	19	0.18	19	4
Puddle core earthfill	9.3	1200	38	1.7	255	19	0.18	19	4
Earthfill with core wall	(<1)	(<130)	(<8)	1.7	255	19	0.18	19	4
Rockfill with core wall	(<1)	(<130)	(<13)	1.7	255	19	0.18	19	4
Hydraulic fill	(<1)	(<130)	(<5)	1.7	255	19	0.18	19	4
All dams	3.5	450	56	1.7	255	19	0.18	19	4

Note: P_{Te}, P_{Tf}, and P_{Tef} are the average frequencies of failure over the life of the dam; P_e, P_f, and P_{ef} are the average annual frequencies of failure. Values in parentheses are based on an assumption of <1 failure.

For a zoned earthfill dam, the following values are taken from Table 1

Piping through Embankment $Pe := 0.000025$

Piping through Foundation $Pf := 0.000019$

Piping from Embankment into Foundation $Pef := 0.000004$

2. Determine numerical weightings from for specific factors and their relative likelihood for piping through the embankment from Table 2:

Table 2. Summary of the weighting factors (values in parentheses) for piping through the embankment mode of failure. Foster et al. (2000)

Factor*	General factors influencing likelihood of failure				
	Much more likely	More likely	Neutral	Less likely	Much less likely
Embankment filters $w_{E(filt)}$		No embankment filter (for dams that usually have filters; refer to text) (2)	Other dam types (1)	Embankment filter present, poor quality (0.2)	Embankment filter present, well designed, and well constructed (0.02)
Core geological origin $w_{E(cgo)}$	Alluvial (1.5)	Aeolian, colluvial (1.25)	Residual, lacustrine, marine, volcanic (1.0)		Glacial (0.5)
Core soil $w_{E(cst)}$	Dispersive clays (5); low-plasticity silts (ML) (2.5); poorly graded and well-graded sands (SP, SW) (2)	Clayey and silty sands (SC, SM) (1.2)	Well-graded and poorly graded gravels (GW, GP) (1.0); high-plasticity silts (MH) (1.0)	Clayey and silty gravels (GC, GM) (0.8); low-plasticity clays (0.8)	High-plasticity clays (CH) (0.3)
Compaction $w_{E(cc)}$	No formal compaction (5)	Rolled, modest control (1.2)	Puddle, hydraulic fill (1.0)		Rolled, good control (0.5)
Conduits $w_{E(con)}$	Conduit through the embankment, many poor details (5)	Conduit through the embankment, some poor details (2)	Conduit through embankment, typical USBR practice (1.0)	Conduit through embankment, including downstream filters (0.8)	No conduit through the embankment (0.5)
Foundation treatment $w_{E(ft)}$	Untreated vertical faces or overhangs in core foundation (2)	Irregularities in foundation or abutment, steep abutments (1.2)		Careful slope modification by cutting, filling with concrete (0.9)	Careful slope modification by cutting, filling with concrete (0.9)
Observations of seepage $w_{E(obs)}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes, seepage emerging on downstream slope (2)	Leakage steady, clear, or not observed (1.0)	Minor leakage (0.7)	Leakage measured none or very small (0.5)
Monitoring and surveillance $w_{E(mon)}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequencies of failure by piping through the embankment depending on zoning type.

Embankment Filters	$Wefilter := 0.02$	Filters present (see Fig. 1), quality not defined.
Core Geological Origin	$Wecgo := 1.5$	Local source material, Surficial Alluvium.
Core Soil Type	$Wecst := 1.2$	Silty sand and Clayey Sand.
Compaction	$Wecc := 0.5$	Compacted according to construction drawing notes.
Conduits	$Wecon := 1.0$	Two 13-foot horsehoe conduits.
Foundation Treatment	$Weft := 0.9$	Cut from natural river valley.
Observations of Seepage	$Weobs := 1.5$	Seepage has been an on-going issue for years.
Monitoring and Surveillance	$Wemon := 0.8$	Piezometers are monitored monthly.

$$WeTOTAL := (Wefilter \cdot Wecgo \cdot Wecst \cdot Wecc \cdot Wecon \cdot Weft \cdot Weobs \cdot Wemon)$$

$WeTOTAL = 0.019$ *Total weighting factor*

$Pe \cdot WeTOTAL = 4.86 \times 10^{-7}$ *Weighted probability for failure through piping in embankment*

3. Determine numerical weightings from for specific factors and their relative likelihood for piping through the foundation from Table 3:

Table 3. Summary of weighting factors (values in parentheses) for piping through the foundation mode of failure. Foster et al. (2000)

	General factors influencing likelihood of failure				
Factor*	Much more likely	More likely	Neutral	Less likely	Much less likely
Filters $w_{F(filt)}$		No foundation filter present when required (1.2)	No foundation filter (1.0)	Foundation filter(s) present (0.8)	
Foundation (below cutoff) $w_{F(fnd)}$	Soil foundation (5)		Rock, clay-infilled or open fractures and (or) erodible rock substance (1.0)	Better rock quality	Rock, closed fractures and non-erodible substance (0.05)
Cutoff (soil foundation) $w_{F(cts)}$		Shallow or no cutoff trench (1.2)	Partially penetrating sheetpile wall or poorly constructed slurry trench wall (1.0)	Upstream blanket, partially penetrating, well-constructed slurry trench wall (0.8)	Partially penetrating deep cutoff trench (0.7)
Cutoff (rock foundation) $w_{F(ctr)}$	Sheetpile wall, poorly constructed diaphragm wall (3)	Well-constructed diaphragm wall (1.5)	Average cutoff trench (1.0)	Well-constructed cutoff trench (0.9)	
Soil geology (below cutoff) $w_{F(sg)}$	Dispersive soils (5); volcanic ash (5)	Residual (1.2)	Aeolian, colluvial, lacustrine, marine (1.0)	Alluvial (0.9)	Glacial (0.5)
Rock geology (below cutoff) $w_{F(rg)}$	Limestone (5); dolomite (3); saline (gypsum) (5); basalt (3)	Tuff (1.5); rhyolite (2); marble (2); quartzite (2)		Sandstone, shale, siltstone, claystone, mudstone, hornfels (0.7); agglomerate, volcanic breccia (0.8)	Conglomerate (0.5); andesite, gabbro (0.5); granite, gneiss (0.2); schist, phyllite, slate (0.5)
Observations of seepage $w_{F(obs)}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes, sand boils (2)	Leakage steady, clear, or not observed (1.0)	Minor leakage (0.7)	Leakage measured none or very small (0.5)
Observations of pore pressures $w_{F(obp)}$	Sudden increases in pressures (up to 10)	Gradually increasing pressures in foundation (2)	High pressures measured in foundation (1.0)		Low pore pressures in foundation (0.8)
Monitoring and surveillance $w_{F(mon)}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequency of failure by piping through the foundation depending on zoning type.

Filters	$Wffilter := 0.8$	Filters present (see Fig. 1), quality not defined.
Foundation Type	$Wffnd := 0.5$	Bedrock.
Cutoff Type	$Wfcut := 1.0$	Core extends to top of bedrock.
Geology	$Wfgeo := 0.9$	Mehrten Formation: Sandstone, Siltstone.
Observations of seepage (or pore pressures)	$Wfobs := 1.5$	Seepage and excess pressures have been an on-going issue for years.
Monitoring and Surveillance	$Wfmon := 0.8$	Piezometers are monitored monthly.

$$WfTOTAL := (Wffilter \cdot Wffnd \cdot Wfcut \cdot Wfgeo \cdot Wfobs \cdot Wfmon)$$

$WfTOTAL = 0.432$ *Total weighting factor*

$Pf \cdot WfTOTAL = 8.208 \times 10^{-6}$ *Weighted probability for failure through piping in embankment*

4. Determine numerical weightings from for specific factors and their relative likelihood for piping from embankment into foundation from Table 4:

Table 4. Summary of weighting factors (values in parentheses) for accidents and failures as a result of piping from the embankment into the foundation. Foster et al. (2000)

Factor*	General factors influencing likelihood of initiation of piping: Much more likely	More likely	Neutral	Less likely	Much less likely
Filters $w_{EF(filt)}$	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)
Foundation cutoff trench $w_{EF(cot)}$	Deep and narrow cutoff trench (1.5)		Average cutoff trench width and depth (1.0)	Shallow or no cutoff trench (0.8)	
Foundation $w_{EF(fnd)}$		Founding on or partly on rock foundations (1.5)			Founding on or partly on soil foundations (0.5)
Erosion-control measures of core foundation $w_{EF(ecm)}$	No erosion-control measures, open-jointed bedrock, or open-work gravels (up to 5)	No erosion-control measures, average foundation conditions (1.2)	No erosion-control measures, good foundation conditions (1.0)	Erosion-control measures present, poor foundations (0.5)	Good to very good erosion-control measures present and good foundation (0.3–0.1)
Grouting of foundations $w_{EF(gr)}$		No grouting on rock foundations (1.3)	Soil foundation only, not applicable (1.0)	Rock foundations grouted (0.8)	
Soil geology types $w_{EF(sg)}$	Colluvial (5)	Glacial (2)		Residual (0.8)	Alluvial, aeolian, lacustrine, marine, volcanic (0.5)
Rock geology types $w_{EF(rg)}$	Sandstone interbedded with shale or limestone (3); limestone, gypsum (2.5)	Dolomite, tuff, quartzite (1.5); rhyolite, basalt, marble (1.2)	Agglomerate, volcanic breccia (1.0); granite, andesite, gabbro, gneiss (1.0)	Sandstone, conglomerate (0.8); schist, phyllite, slate, hornfels (0.6)	Shale, siltstone, mudstone, claystone, (0.2)
Core geological origin $w_{EF(cgo)}$	Alluvial (1.5)	Aeolian, colluvial (1.25)	Residual, lacustrine, marine, volcanic (1.0)		Glacial (0.5)
Core soil type $w_{EF(cst)}$	Dispersive clays (5); low-plasticity silts (ML) (2.5); poorly graded and well-graded sands (SP, SW) (2)	Clayey and silty sands (SC, SM) (1.2)	Well-graded and poorly graded gravels (GW, GP) (1.0); high-plasticity silts (MH) (1.0)	Clayey and silty gravels (GC, GM) (0.8); low-plasticity clays (CL) (0.8)	High-plasticity clays (CH) (0.3)
Core compaction $w_{EF(cc)}$	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)
Foundation treatment $w_{EF(ft)}$	Untreated vertical faces or overhangs in core foundation (1.5)	Irregularities in foundation or abutment, steep abutments (1.1)		Careful slope modification by cutting, filling with concrete (0.9)	Careful slope modification by cutting, filling with concrete (0.9)
Observations of seepage $w_{EF(obs)}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes (2)	Leakage steady, clear, or not monitored (1.0)	Minor leakage (0.7)	No or very small leakage measured (0.5)
Monitoring and surveillance $w_{EF(mon)}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequency of failure by piping from the embankment into the foundation depending on zoning type.

Filters	Weffilter := 1.0	Filters present (see Fig. 1), quality not defined.
Foundation Cutoff Trench	Wefcut := 0.8	Shallow Core extends to top of bedrock.

Foundation Type	$Wefft := 1.5$	Bedrock.
Erosion Control Measures of Core Foundation	$Wefecm := 1.2$	No erosion control measures, surface prepared prior to embankment construction.
Grouting of Foundations	$Wefgrout := 0.8$	Grouted bedrock foundation.
Geology	$Wefgeo := 0.8$	Mehrten Formation: Sandstone, Siltstone.
Core Geological Origin	$Wefcgeo := 1.5$	Local source material, Surficial Alluvium.
Core Soil Type	$Wefcst := 1.2$	Silty sand and Clayey Sand.
Core Compaction	$Wefcc := 1.0$	Compacted according to construction drawing notes.
Foundation Treatment	$Wefftr := 1.0$	Cut from natural river valley.
Observations of Seepage	$Wefobs := 1.5$	Seepage has been an on-going issue for years.
Monitoring and Surveillance	$Wefmon := 0.8$	Piezometers are monitored monthly.

$WefTOTAL = 1.991$ *Total weighting factor (product of all specific factors)*

$Pef \cdot WefTOTAL = 7.963 \times 10^{-6}$ *Weighted probability for failure, piping from embankment into foundation*

5. Sum of the primary failure modes yields the overall failure probability due to piping:

Overall Annual Probability of Failure by piping:

$$Pp := (Pe \cdot WeTOTAL) + (Pf \cdot WfTOTAL) + (Pef \cdot WefTOTAL)$$

$$Pp = 1.666 \times 10^{-5}$$

Evaluation of Annual Potential of Failure for Dike No. 2

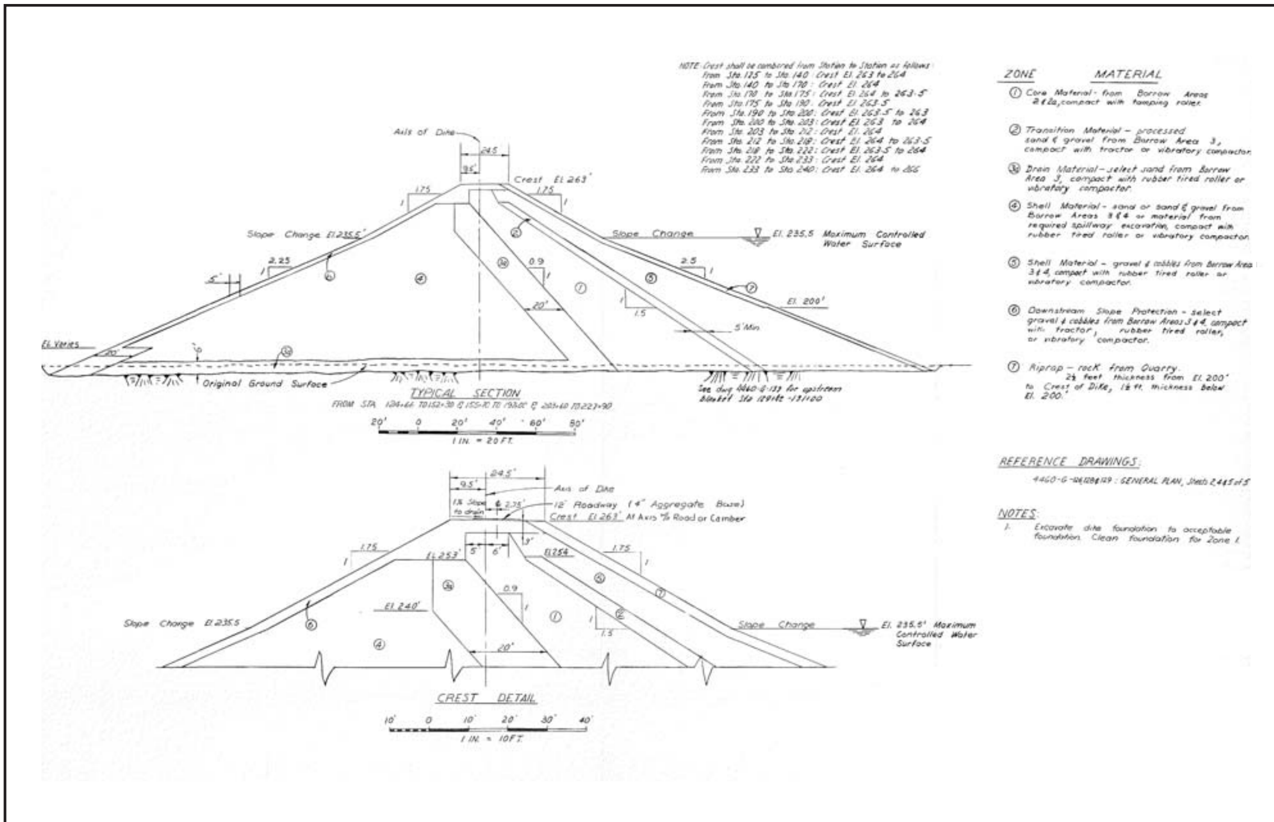
STATEMENT: Assess the annual probability of failure for Dike No. 2 due to piping.

REFERENCES:

1. M. Foster, R. Fell, and M. Spannagle, A Method for Assessing the Relative Likelihood of Failure of Embankment Dams by Piping, Canadian Geotechnical Journal, 2000, Vol. 37 No.5.

SCOPE:

1. Quantify the necessary components of the Dike No. 2 and their current condition.
2. Apply weighting factors for specific factors associated with failure of the Dike due to piping.



Dike No. 2 Typical Cross-Section (See Figure 2-2 of Report for Full-Size Figure)

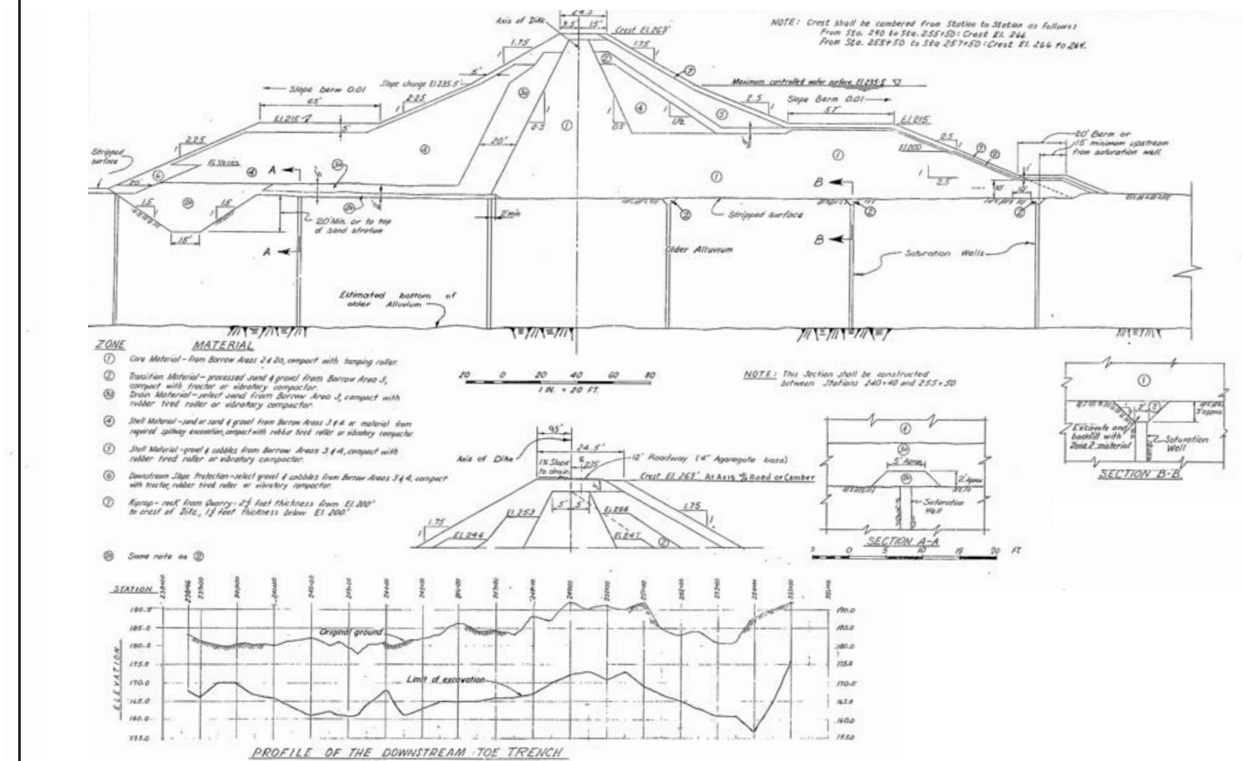


Figure 2 - Dike No. 2 Deep Cross-Section (See Figure 3 of Memorandum for Full-Size Figure)

1. Determine the average annual probabilities of failure for each of three primary modes of failure:

i - piping through embankment
ii - piping through foundation
iii - piping from the embankment into the foundation

Dike was constructed in 1964 (older than 5 years).

Table 1. Average historic frequency of failure of embankment dams by mode of failure and dam zoning. Foster et al. (2000)

Zoning category	Embankment: Average P_{Tc} (10^{-3})	Embankment: Average annual P_e (10^{-6}) First 5 years operation	Embankment: Average annual P_e (10^{-6}) After 5 years operation	Foundation: Average P_{Tf} (10^{-3})	Foundation: Average annual P_f (10^{-6}) First 5 years operation	Foundation: Average annual P_f (10^{-6}) After 5 years operation	Embankment into foundation: Average P_{Tef} (10^{-3})	Embankment into foundation: Average annual P_{ef} (10^{-6}) First 5 years operation	Embankment into foundation: Average annual P_{ef} (10^{-6}) After 5 years operation
Homogeneous earthfill	16	2080	190	1.7	255	19	0.18	19	4
Earthfill with filter	1.5	190	37	1.7	255	19	0.18	19	4
Earthfill with rock toe	8.9	1160	160	1.7	255	19	0.18	19	4
Zoned earthfill	1.2	160	25	1.7	255	19	0.18	19	4
Zoned earth and rockfill	1.2	150	24	1.7	255	19	0.18	19	4
Central core earth and rockfill	(<1)	(<140)	(<34)	1.7	255	19	0.18	19	4
Concrete face earthfill	5.3	690	75	1.7	255	19	0.18	19	4
Concrete face rockfill	(<1)	(<130)	(<17)	1.7	255	19	0.18	19	4
Puddle core earthfill	9.3	1200	38	1.7	255	19	0.18	19	4
Earthfill with core wall	(<1)	(<130)	(<8)	1.7	255	19	0.18	19	4
Rockfill with core wall	(<1)	(<130)	(<13)	1.7	255	19	0.18	19	4
Hydraulic fill	(<1)	(<130)	(<5)	1.7	255	19	0.18	19	4
All dams	3.5	450	56	1.7	255	19	0.18	19	4

Note: P_{Te}, P_{Tf}, and P_{Tef} are the average frequencies of failure over the life of the dam; P_e, P_f, and P_{ef} are the average annual frequencies of failure. Values in parentheses are based on an assumption of <1 failure.

For a zoned earthfill dam, the following values are taken from Table 1.

Piping through Embankment $Pe := 0.000025$

Piping through Foundation $Pf := 0.000019$

Piping from Embankment into Foundation $Pef := 0.000004$

2. Determine numerical weightings from for specific factors and their relative likelihood for piping through the embankment from Table 2:

Table 2. Summary of the weighting factors (values in parentheses) for piping through the embankment mode of failure. Foster et al. (2000)

	General factors influencing likelihood of failure				
Factor*	Much more likely	More likely	Neutral	Less likely	Much less likely
Embankment filters $w_{E(filt)}$		No embankment filter (for dams that usually have filters; refer to text) (2)	Other dam types (1)	Embankment filter present, poor quality (0.2)	Embankment filter present, well designed, and well constructed (0.02)
Core geological origin $w_{E(cgo)}$	Alluvial (1.5)	Aeolian, colluvial (1.25)	Residual, lacustrine, marine, volcanic (1.0)		Glacial (0.5)
Core soil $w_{E(cst)}$	Dispersive clays (5); low-plasticity silts (ML) (2.5); poorly graded and well-graded sands (SP, SW) (2)	Clayey and silty sands (SC, SM) (1.2)	Well-graded and poorly graded gravels (GW, GP) (1.0); high-plasticity silts (MH) (1.0)	Clayey and silty gravels (GC, GM) (0.8); low-plasticity clays (0.8)	High-plasticity clays (CH) (0.3)
Compaction $w_{E(cc)}$	No formal compaction (5)	Rolled, modest control (1.2)	Puddle, hydraulic fill (1.0)		Rolled, good control (0.5)
Conduits $w_{E(con)}$	Conduit through the embankment, many poor details (5)	Conduit through the embankment, some poor details (2)	Conduit through embankment, typical USBR practice (1.0)	Conduit through embankment, including downstream filters (0.8)	No conduit through the embankment (0.5)
Foundation treatment $w_{E(ft)}$	Untreated vertical faces or overhangs in core foundation (2)	Irregularities in foundation or abutment, steep abutments (1.2)		Careful slope modification by cutting, filling with concrete (0.9)	Careful slope modification by cutting, filling with concrete (0.9)
Observations of seepage $w_{E(obs)}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes, seepage emerging on downstream slope (2)	Leakage steady, clear, or not observed (1.0)	Minor leakage (0.7)	Leakage measured none or very small (0.5)
Monitoring and surveillance $w_{E(mon)}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequencies of failure by piping through the embankment depending on zoning type.

Embankment Filters	$Wefilter := 0.2$	Filters present (see Fig. 1), quality not defined.
Core Geological Origin	$Wecgo := 1.0$	Local source material, Surficial Alluvium.
Core Soil Type	$Wecst := 1.2$	Silty sand and Clayey Sand.
Compaction	$Wecc := 0.5$	Compacted according to construction drawing notes.
Conduits	$Wecon := 0.5$	No conduits.
Foundation Treatment	$Weft := 1.0$	Cut from natural topography.
Observations of Seepage	$Weobs := 2.0$	Seepage has been an on-going issue for years.
Monitoring and Surveillance	$Wemon := 0.8$	Piezometers are monitored monthly.

$$WeTOTAL := (Wefilter \cdot Wecgo \cdot Wecst \cdot Wecc \cdot Wecon \cdot Weft \cdot Weobs \cdot Wemon)$$

$WeTOTAL = 0.096$ *Total weighting factor*

$Pe \cdot WeTOTAL = 2.4 \times 10^{-6}$ *Weighted probability for failure through piping in embankment*

3. Determine numerical weightings from for specific factors and their relative likelihood for piping through the foundation from Table 3:

Table 3. Summary of weighting factors (values in parentheses) for piping through the foundation mode of failure. Foster et al. (2000)

	General factors influencing likelihood of failure				
Factor*	Much more likely	More likely	Neutral	Less likely	Much less likely
Filters $w_{F(filt)}$		No foundation filter present when required (1.2)	No foundation filter (1.0)	Foundation filter(s) present (0.8)	
Foundation (below cutoff) $w_{F(fnd)}$	Soil foundation (5)		Rock, clay-infilled or open fractures and (or) erodible rock substance (1.0)	Better rock quality	Rock, closed fractures and non-erodible substance (0.05)
Cutoff (soil foundation) $w_{F(cts)}$		Shallow or no cutoff trench (1.2)	Partially penetrating sheetpile wall or poorly constructed slurry trench wall (1.0)	Upstream blanket, partially penetrating, well-constructed slurry trench wall (0.8)	Partially penetrating deep cutoff trench (0.7)
Cutoff (rock foundation) $w_{F(ctr)}$	Sheetpile wall, poorly constructed diaphragm wall (3)	Well-constructed diaphragm wall (1.5)	Average cutoff trench (1.0)	Well-constructed cutoff trench (0.9)	
Soil geology (below cutoff) $w_{F(sg)}$	Dispersive soils (5); volcanic ash (5)	Residual (1.2)	Aeolian, colluvial, lacustrine, marine (1.0)	Alluvial (0.9)	Glacial (0.5)
Rock geology (below cutoff) $w_{F(rg)}$	Limestone (5); dolomite (3); saline (gypsum) (5); basalt (3)	Tuff (1.5); rhyolite (2); marble (2); quartzite (2)		Sandstone, shale, siltstone, claystone, mudstone, hornfels (0.7); agglomerate, volcanic breccia (0.8)	Conglomerate (0.5); andesite, gabbro (0.5); granite, gneiss (0.2); schist, phyllite, slate (0.5)
Observations of seepage $w_{F(obs)}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes, sand boils (2)	Leakage steady, clear, or not observed (1.0)	Minor leakage (0.7)	Leakage measured none or very small (0.5)
Observations of pore pressures $w_{F(obp)}$	Sudden increases in pressures (up to 10)	Gradually increasing pressures in foundation (2)	High pressures measured in foundation (1.0)		Low pore pressures in foundation (0.8)
Monitoring and surveillance $w_{F(mon)}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequency of failure by piping through the foundation depending on zoning type.

Filters	$Wffilter := 1$	Filters present (see Fig. 1), quality not defined.
Foundation Type	$Wffnd := 1$	Bedrock.
Cutoff Type	$Wfcut := 1$	Core extends to top of bedrock, upstream toe slurry trench to bedrock.
Geology	$Wfgeo := 0.7$	Mehrten Formation: Sandstone, Siltstone.
Observations of seepage (or pore pressures)	$Wfobs := 2.0$	Seepage and excess pressures have been an on-going issue for years.
Monitoring and Surveillance	$Wfmon := 0.8$	Piezometers are monitored monthly.

$$WfTOTAL := (Wffilter \cdot Wffnd \cdot Wfcut \cdot Wfgeo \cdot Wfobs \cdot Wfmon)$$

$WfTOTAL = 1.12$ *Total weighting factor*

$Pf \cdot WfTOTAL = 2.128 \times 10^{-5}$ *Weighted probability for failure through piping in embankment*

4. Determine numerical weightings from for specific factors and their relative likelihood for piping from embankment into foundation from Table 4:

Table 4. Summary of weighting factors (values in parentheses) for accidents and failures as a result of piping from the embankment into the foundation. Foster et al. (2000)

	General factors influencing likelihood of initiation of piping				
Factor*	Much more likely	More likely	Neutral	Less likely	Much less likely
Filters $w_{EF(filt)}$	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)
Foundation cutoff trench $w_{EF(cot)}$	Deep and narrow cutoff trench (1.5)		Average cutoff trench width and depth (1.0)	Shallow or no cutoff trench (0.8)	
Foundation $w_{EF(fnd)}$		Founding on or partly on rock foundations (1.5)			Founding on or partly on soil foundations (0.5)
Erosion-control measures of core foundation $w_{EF(ecm)}$	No erosion-control measures, open-jointed bedrock, or open-work gravels (up to 5)	No erosion-control measures, average foundation conditions (1.2)	No erosion-control measures, good foundation conditions (1.0)	Erosion-control measures present, poor foundations (0.5)	Good to very good erosion-control measures present and good foundation (0.3–0.1)
Grouting of foundations $w_{EF(gr)}$		No grouting on rock foundations (1.3)	Soil foundation only, not applicable (1.0)	Rock foundations grouted (0.8)	
Soil geology types $w_{EF(sg)}$	Colluvial (5)	Glacial (2)		Residual (0.8)	Alluvial, aeolian, lacustrine, marine, volcanic (0.5)
Rock geology types $w_{EF(rg)}$	Sandstone interbedded with shale or limestone (3); limestone, gypsum (2.5)	Dolomite, tuff, quartzite (1.5); rhyolite, basalt, marble (1.2)	Agglomerate, volcanic breccia (1.0); granite, andesite, gabbro, gneiss (1.0)	Sandstone, conglomerate (0.8); schist, phyllite, slate, hornfels (0.6)	Shale, siltstone, mudstone, claystone, (0.2)
Core geological origin $w_{EF(cgo)}$	Alluvial (1.5)	Aeolian, colluvial (1.25)	Residual, lacustrine, marine, volcanic (1.0)		Glacial (0.5)
Core soil type $w_{EF(cst)}$	Dispersive clays (5); low-plasticity silts (ML) (2.5); poorly graded and well-graded sands (SP, SW) (2)	Clayey and silty sands (SC, SM) (1.2)	Well-graded and poorly graded gravels (GW, GP) (1.0); high-plasticity silts (MH) (1.0)	Clayey and silty gravels (GC, GM) (0.8); low-plasticity clays (CL) (0.8)	High-plasticity clays (CH) (0.3)
Core compaction $w_{EF(cc)}$	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)
Foundation treatment $w_{EF(ft)}$	Untreated vertical faces or overhangs in core foundation (1.5)	Irregularities in foundation or abutment, steep abutments (1.1)		Careful slope modification by cutting, filling with concrete (0.9)	Careful slope modification by cutting, filling with concrete (0.9)
Observations of seepage $w_{EF(obs)}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes (2)	Leakage steady, clear, or not monitored (1.0)	Minor leakage (0.7)	No or very small leakage measured (0.5)
Monitoring and surveillance $w_{EF(mon)}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequency of failure by piping from the embankment into the foundation depending on zoning type.

Filters	Weffilter := 1.0	Filters present (see Fig. 1), quality not defined.
Foundation Cutoff Trench	Wefcut := 0.8	Core extends to top of bedrock.

Foundation Type	$Wefft := 1.0$	Bedrock and Soil.
Erosion Control Measures of Core Foundation	$Wefecm := 1.2$	No erosion control measures.
Grouting of Foundations	$Wefgrout := 1.3$	Foundation not grouted.
Geology	$Wefgeo := 0.8$	Mehrten Formation: Sandstone, Siltstone.
Core Geological Origin	$Wefcgeo := 1.0$	Local source material, Surficial Alluvium.
Core Soil Type	$Wefcst := 1.2$	Silty sand and Clayey Sand.
Core Compaction	$Wefcc := 1.0$	Compacted according to construction drawing notes.
Foundation Treatment	$Wefftr := 1.0$	Cut from natural topography.
Observations of Seepage	$Wefobs := 2.0$	Seepage and excess pressures have been an on-going issue for years.
Monitoring and Surveillance	$Wefmon := 0.8$	Piezometers are monitored monthly.

$WefTOTAL = 1.917$ *Total weighting factor (product of all specific factors)*

$Pef \cdot WefTOTAL = 7.668 \times 10^{-6}$ *Weighted probability for failure, piping from embankment into foundation*

5. Sum of the three primary failure modes yields the overall failure probability due to piping:

Annual Probability of Failure of the Embankment Dam

$$Prob := (Pe \cdot WeTOTAL) + (Pf \cdot WfTOTAL) + (Pef \cdot WefTOTAL)$$

$$Prob = 3.135 \times 10^{-5}$$

APPENDIX C
DESCRIPTION OF MEHRTEN AND VALLEY SPRINGS FORMATIONS

The Oligocene- to Miocene-age Valley Springs Formation (20-25 million years old) consists of interbedded fluvial deposits (siltstone, sandstone, and conglomerate) and rhyolitic tuffs. Beds within the Valley Springs Formation are relatively continuous laterally, and are locally permeable.

The Miocene to Pliocene-age Mehrten Formation (4.6 to 10 million years old) consists of coarse- to fine-grained volcaniclastic sediments, volcanic agglomerates, tuffs, and local lava flows deposited in former stream channels. The Mehrten deposits are resistant to erosion and Mehrten-filled Cenozoic-age stream channels now form discontinuous linear ridges that strike roughly perpendicular to the regional structure. Volcanic sandstones comprise more than 50 percent of the formation at the site of the Main Dam, but well cemented siltstones and claystones are also present. Volcanic mudflow deposits are typically hard, impervious, and contain angular volcanic rock fragments. Conglomerates consist primarily of rounded andesitic rock fragments varying from pebble to cobble size within a fine- to medium-grained sandstone matrix. Beds within the Mehrten Formation tend to be lenticular, but some permeable layers can persist for distances up to a few hundred feet.

The volcanic debris flow (mudflow) that caps the upper abutments at the Main Dam is erosionally-resistant, massive (i.e., very thick-bedded and mostly unfractured), and strong. It contains angular gravel to boulder-sized blocks of hard and strong andesite that are encased within a dense, moderately hard and well-indurated/cemented, sandy silt matrix. This unit appears to be relatively impermeable. The Main Dam spillway channel was excavated through this massive strong debris flow deposit and that excavation required blasting.

Within the local vicinity of the project, the occurrence of this debris flow unit appears to be limited to the uppermost ridge tops. Detailed study of the Mehrten in this region (Wagner, 1981) indicates that in this area the debris flow is relatively thin (commonly less than 20 feet) and underlain by thick fluvial deposits of andesitic detritus. At the Main Dam, the basal surface of the debris flow appears to drop fairly steeply in a downstream (westerly) direction, suggesting that the flow may have filled a southerly-oriented paleochannel; the thickness of the flow at the left abutment appears to be on the order of about 40 feet.

The debris flow unit is commonly underlain by a comparatively thin conglomerate deposit (generally less than 10 feet thick where observed along the reservoir shoreline below Dike 3). This clast-supported conglomerate contains rounded, hard gravel and cobbles (mostly andesite), and typically appears well indurated and strong with a tightly packed, fine grained matrix. Locally, however, the fine-grained matrix material is absent, and the conglomerate appears open and permeable as shown on the photo in Figure C-1. Locally high grout takes along the left end of the Main Dam grout curtain may have occurred within such open conglomerate layers beneath the debris flow capping the left abutment.

The vast majority of the Mehrten Formation underlying the embankments consists of interbedded sandstones, siltstones and claystones. Bechtel logs indicate that these layers vary from well-cemented to uncemented, with correspondingly great variation in permeability. Observations of the upper portions of these deposits, exposed along the reservoir shoreline, indicate that at least some of these layers are fine-grained sands and silty sands that are, although dense, relatively soft (as rock) and probably relatively erodible. Animal burrowing along less-cemented sandstone layers attests to the local erodibility of some Mehrten layers as shown in the photo in Figure C-2.

The Mehrten deposits commonly occur as thick bedded to massive layers that are very little fractured. Only one significant joint set was noted during the recent 2008 field investigations and during the design studies and construction by Bechtel; this set consists of near-vertical to vertical, very widely spaced joints that trend N20W to N40W. The Main Dam grout curtain was designed with angled holes to intercept these joints. Where presently exposed, these joints appear to have great lateral and vertical continuity (greater than 50 feet), but generally appear to be very narrow to tight in aperature. A faint moisture/spring line (green vegetation) trending along what appears to be a N30W 85W joint, on the downstream left abutment groin was noted during reconnaissance mapping in June 2008 and may represent localized reservoir seepage along this major joint set.

REFERENCE

Wagner, H.M., 1981, *Geochronology of the Mehrten Formation in Stanislaus County*, PhD Dissertation, U.C. Riverside, June 1981.

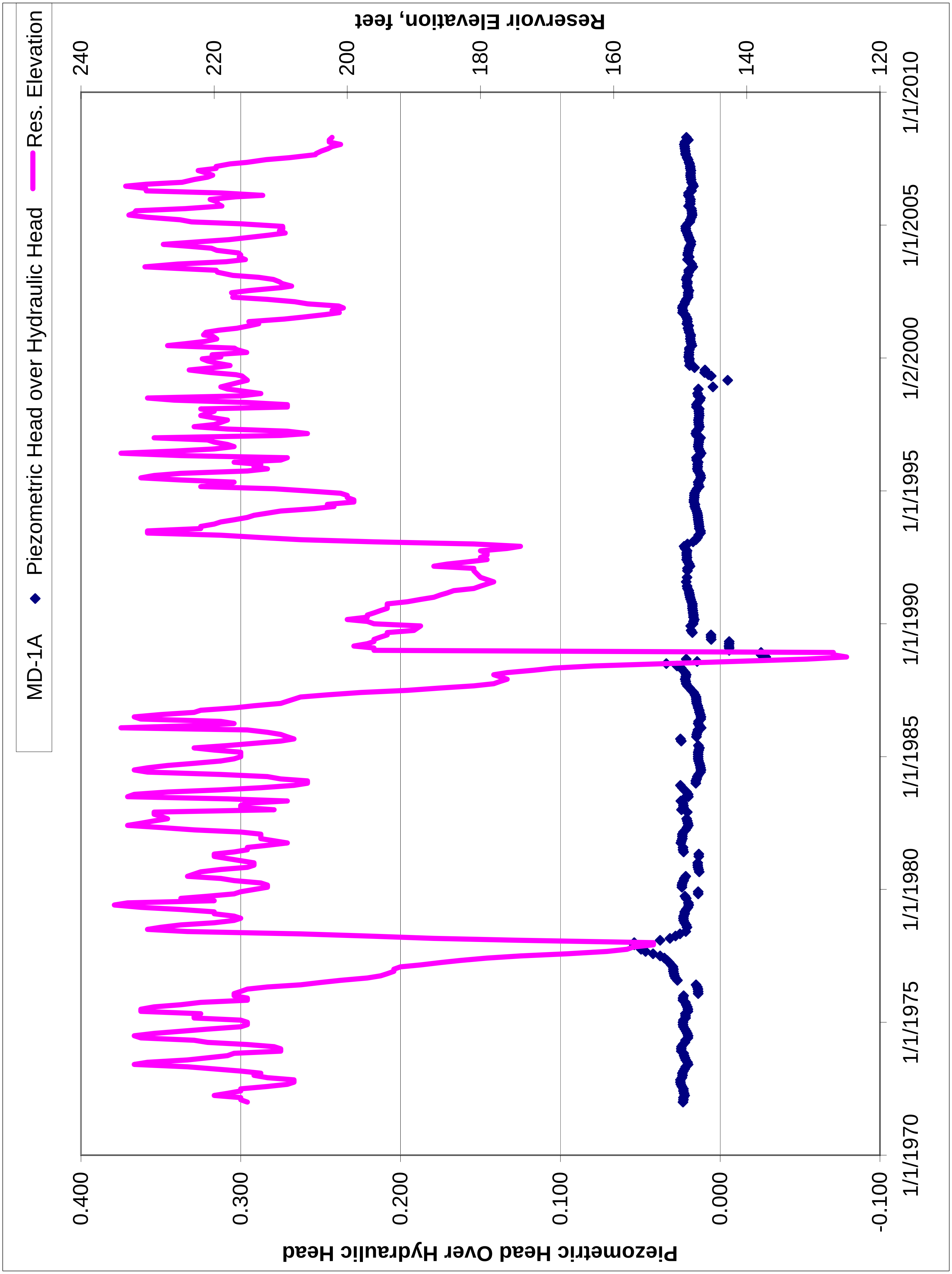
Figure C-1 Locally Open Mehrten Conglomerate, with Little Fine-Grained Matrix, Underlying Volcanic Debris Flow; Reservoir Bank below Dike 3.

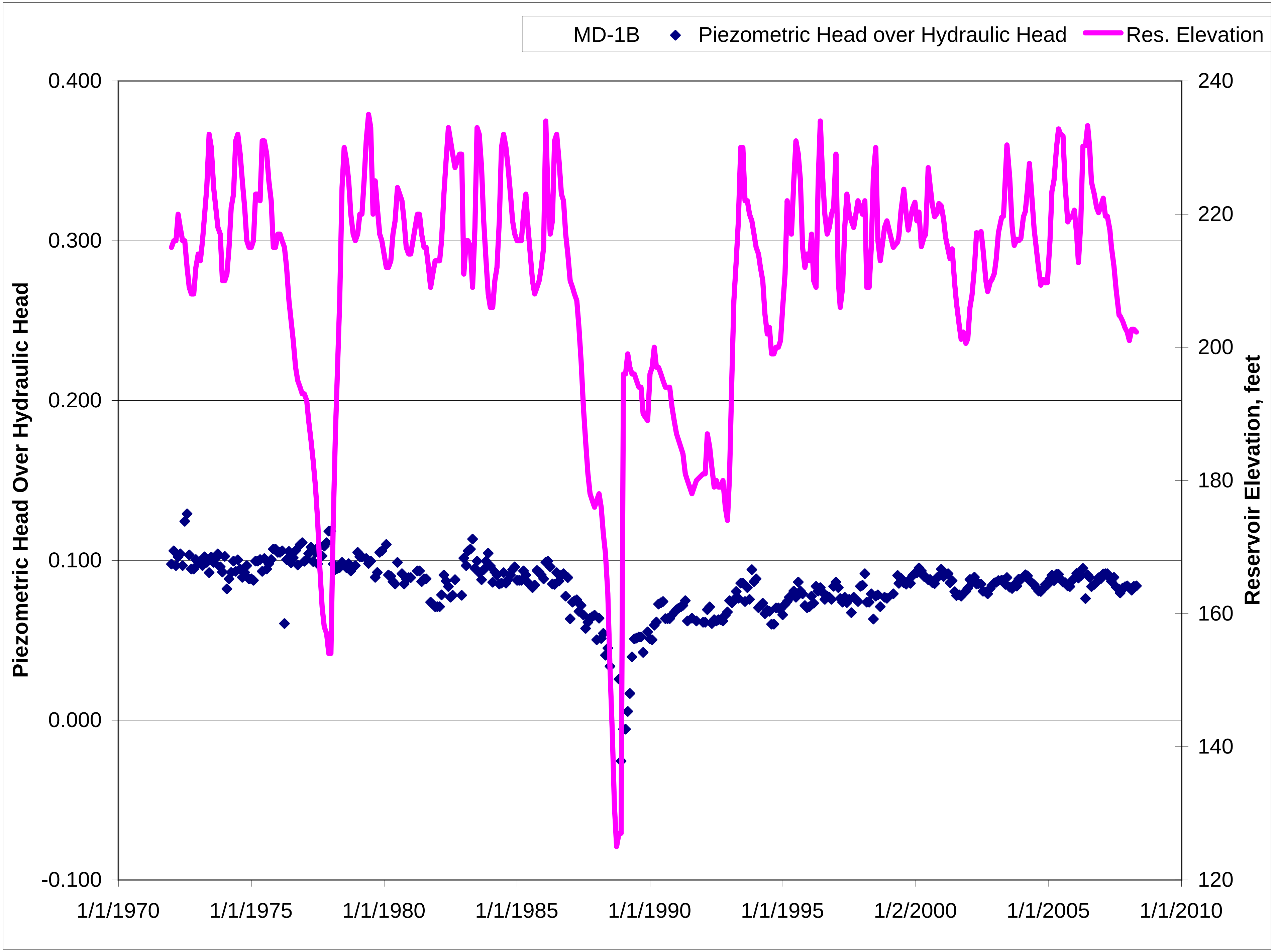
Figure C-2 Burrow Cavities along Weakly Cemented Mehrten Sandstone Layers; Main Dam, Upstream of Right Abutment

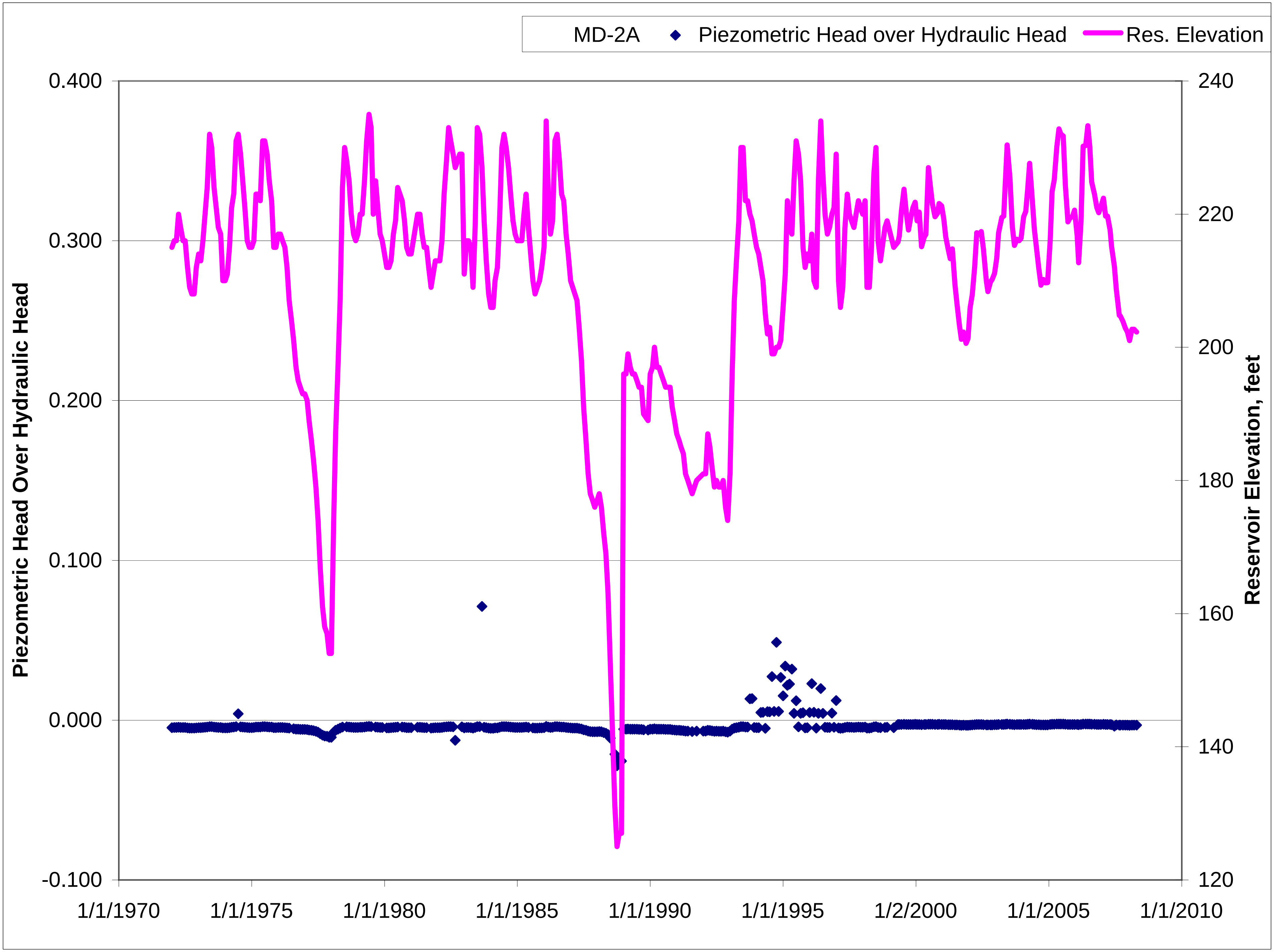
APPENDIX D
HEAD RATIOS AND RESERVOIR LEVEL VS. TIME

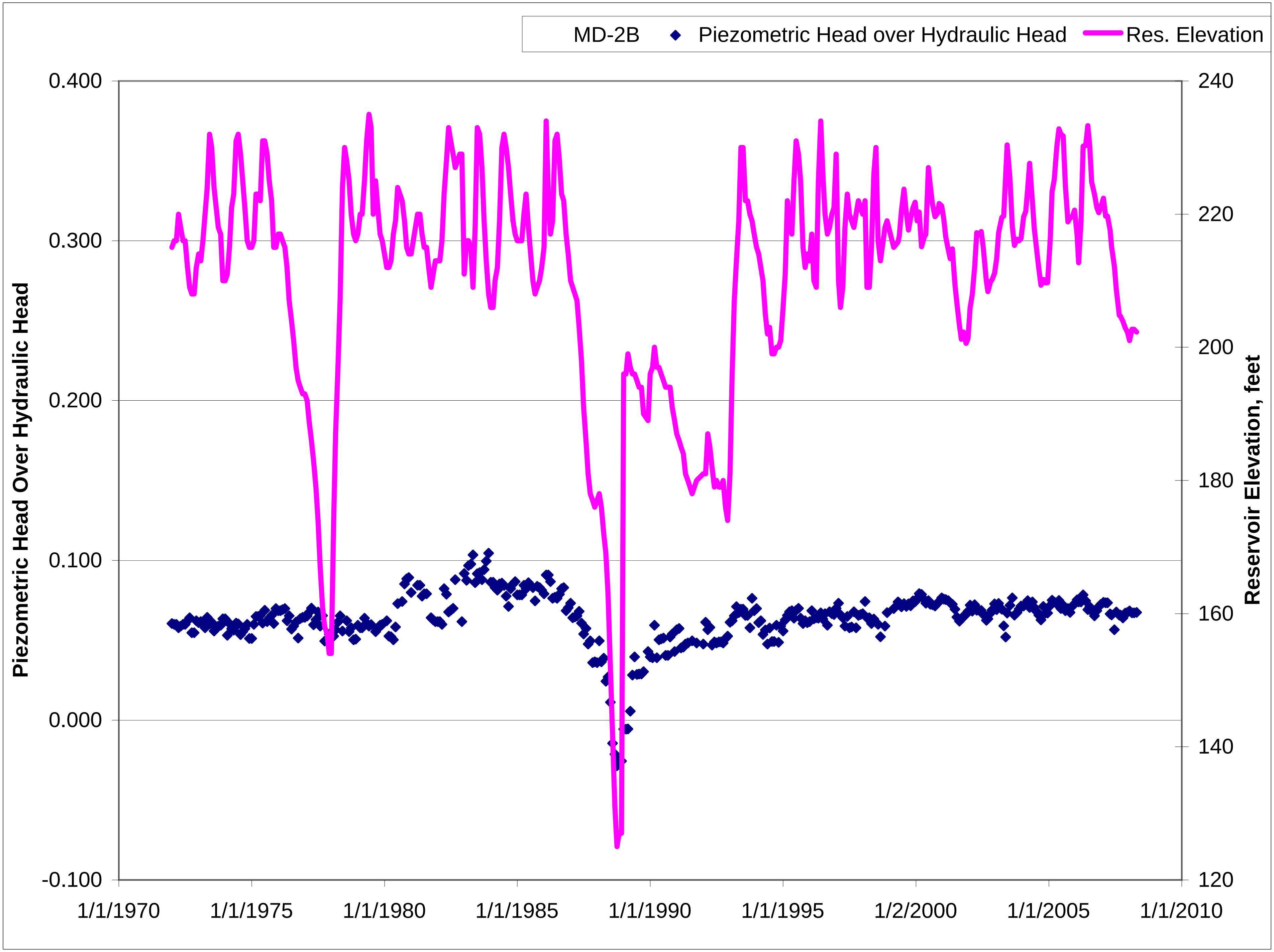
Page

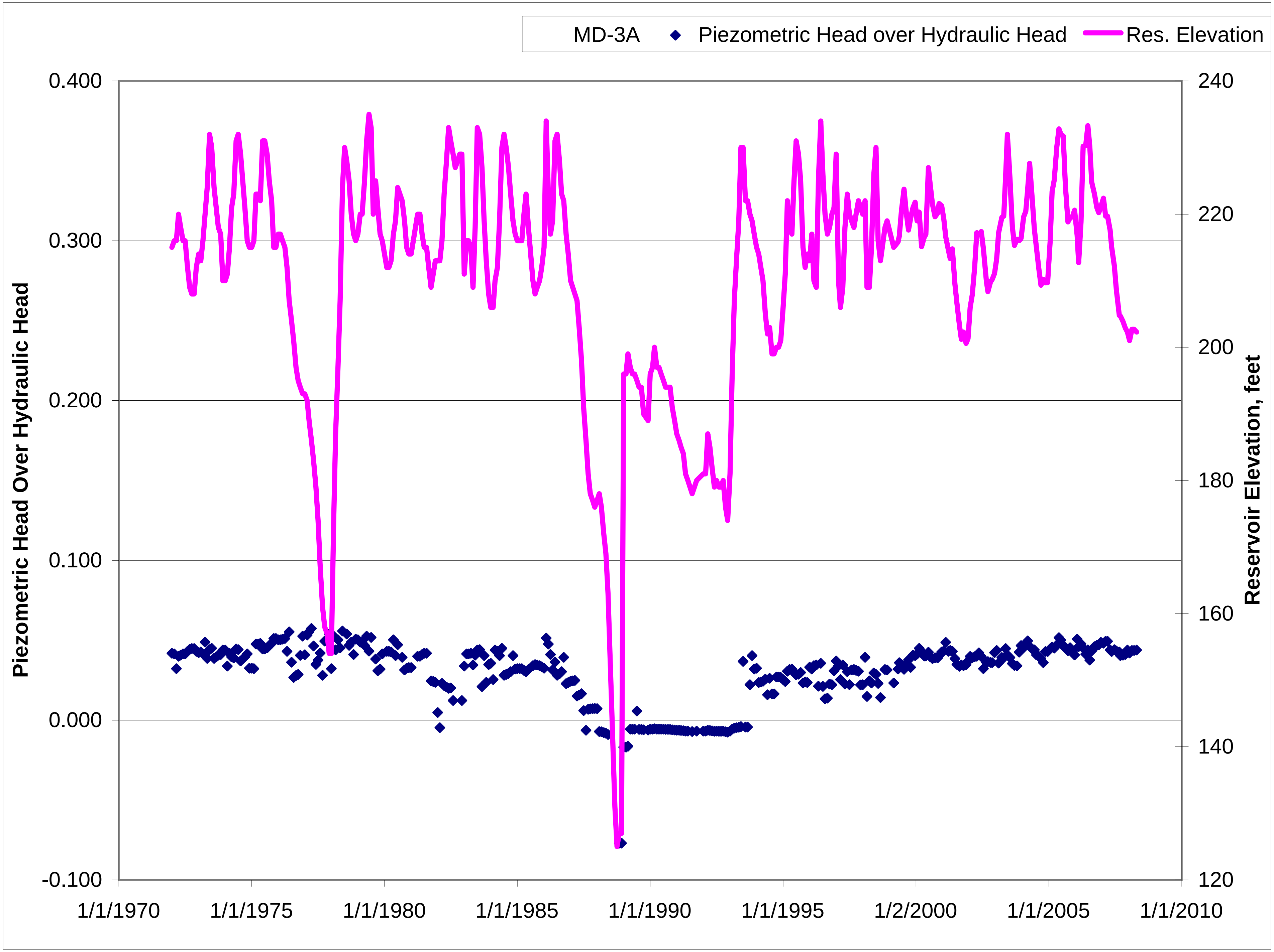
Page

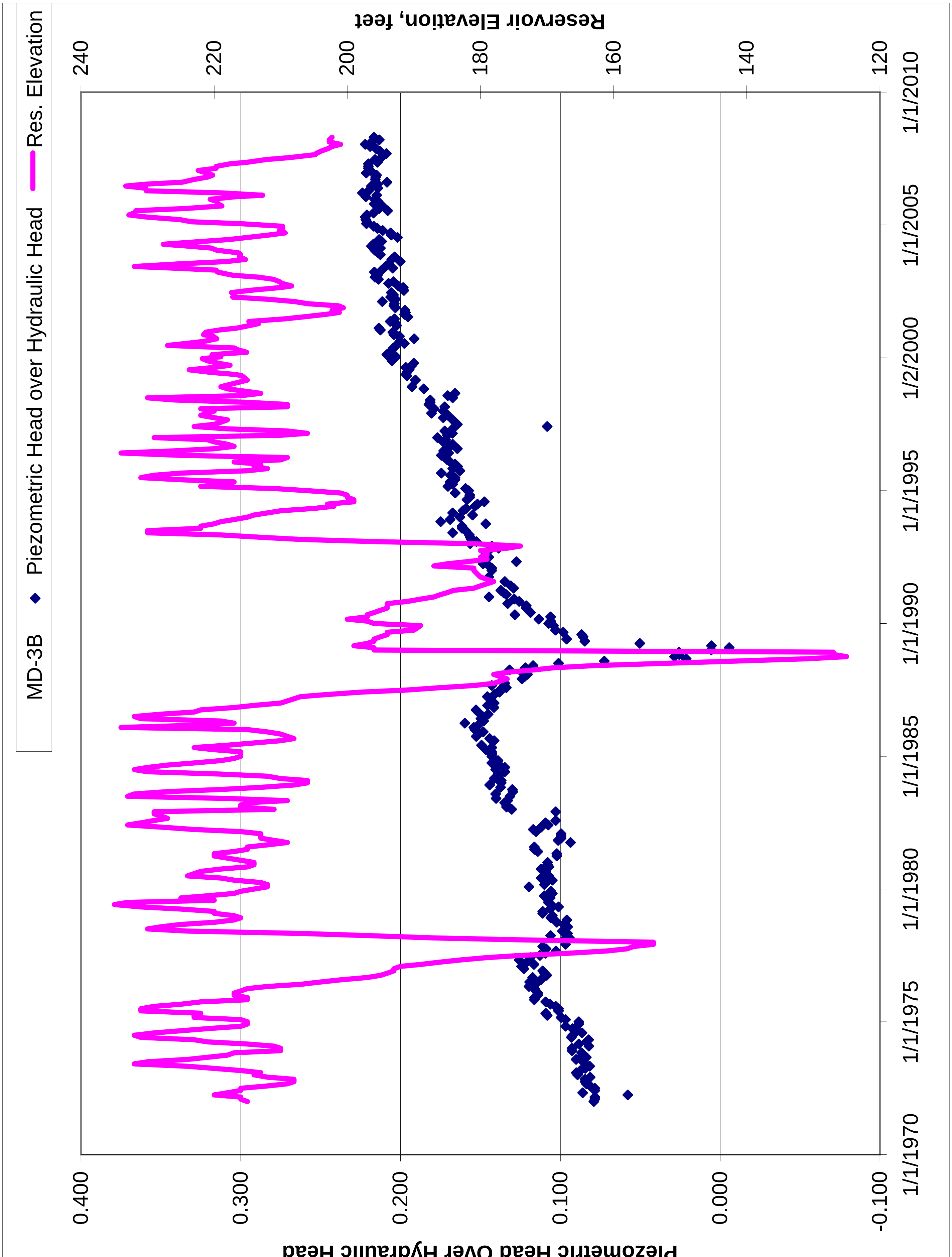

MD-1A
Piezometric Head over Hydraulic Head
Res. Elevation
Reservoir Elevation, feet
Piezometric Head Over Hydraulic Head
240
220
200
180
160
140
120
0.400
0.300
0.200
0.100
0.000
-0.100
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

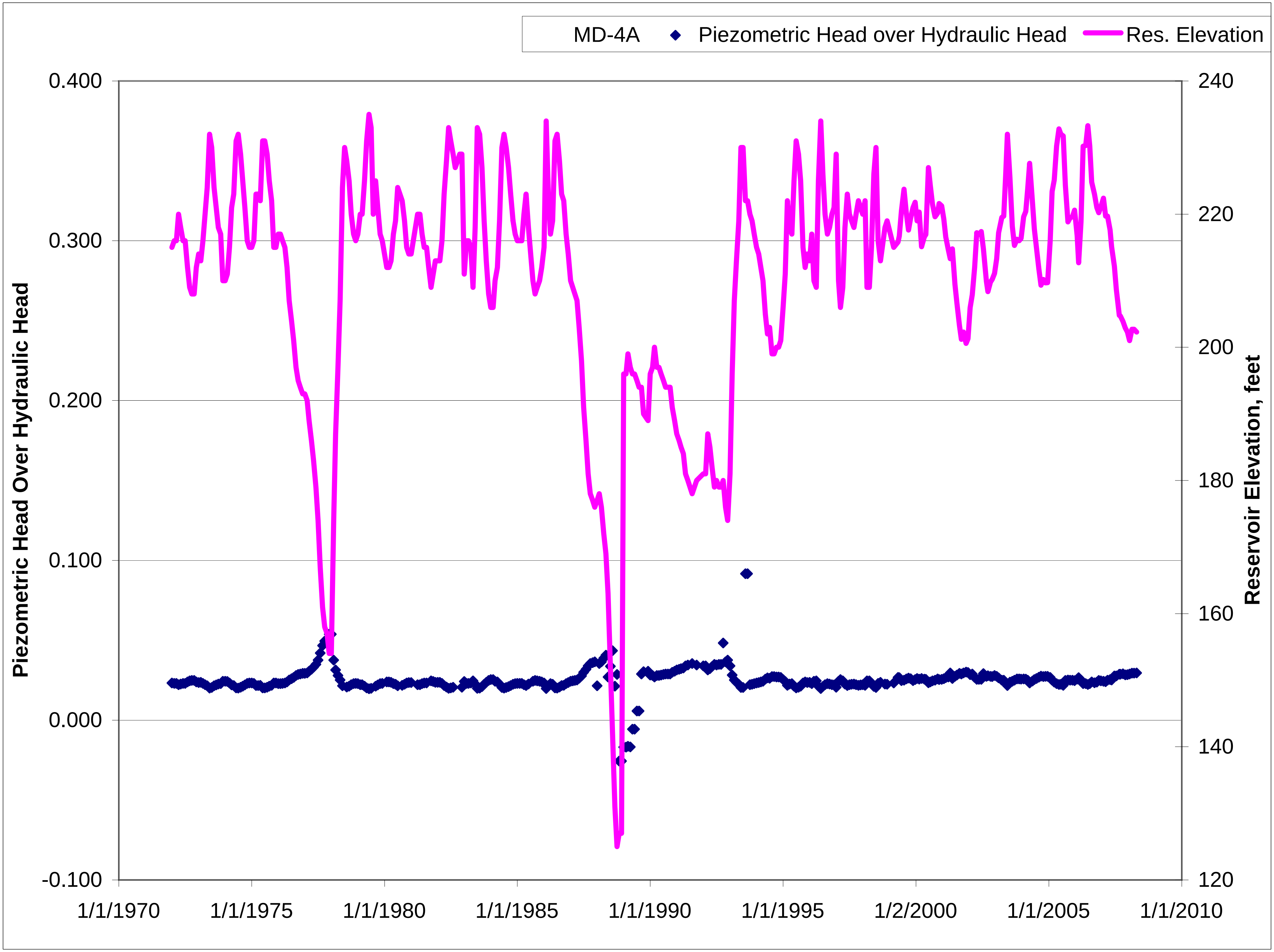

MD-1B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

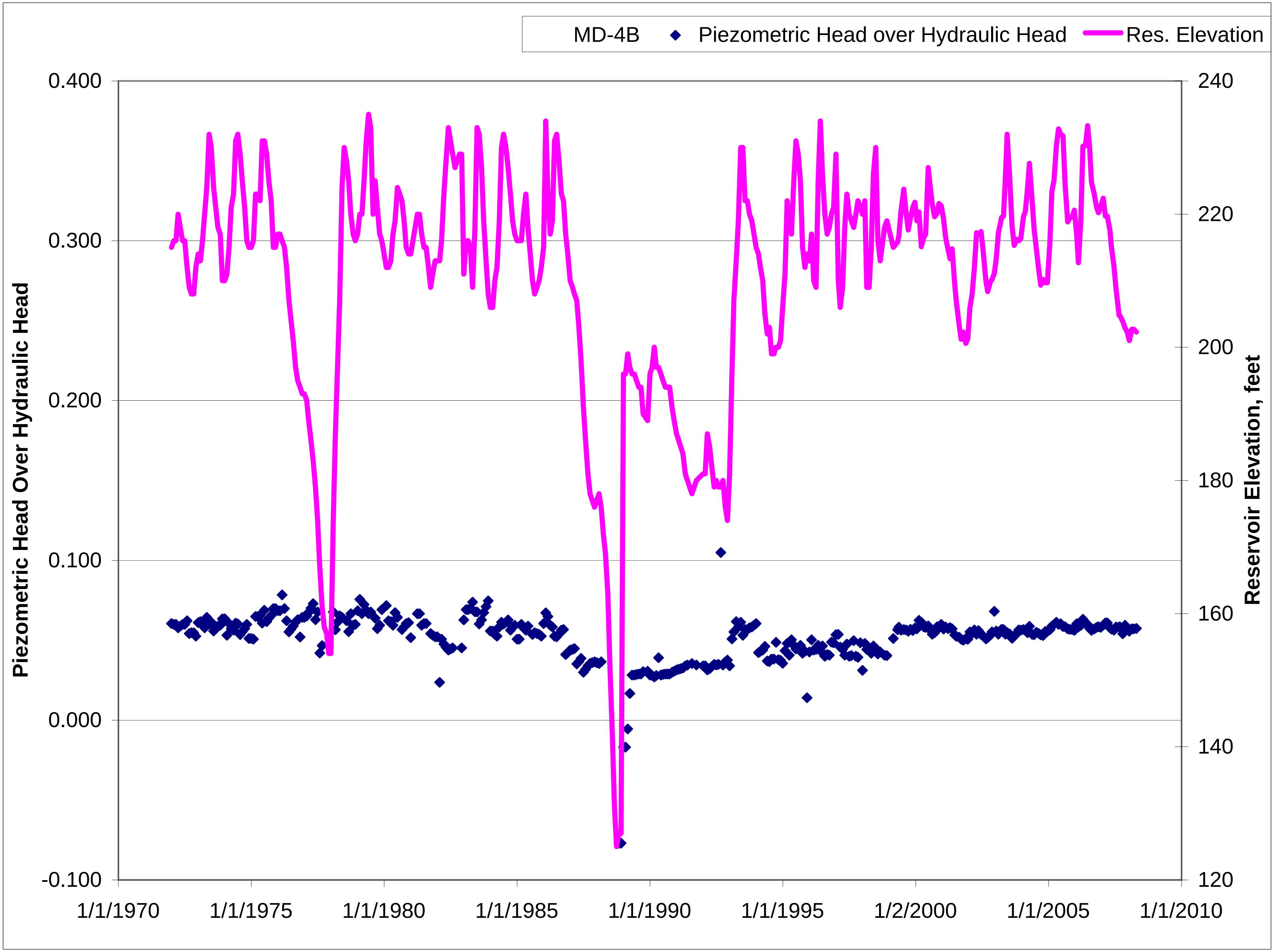

MD-2A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

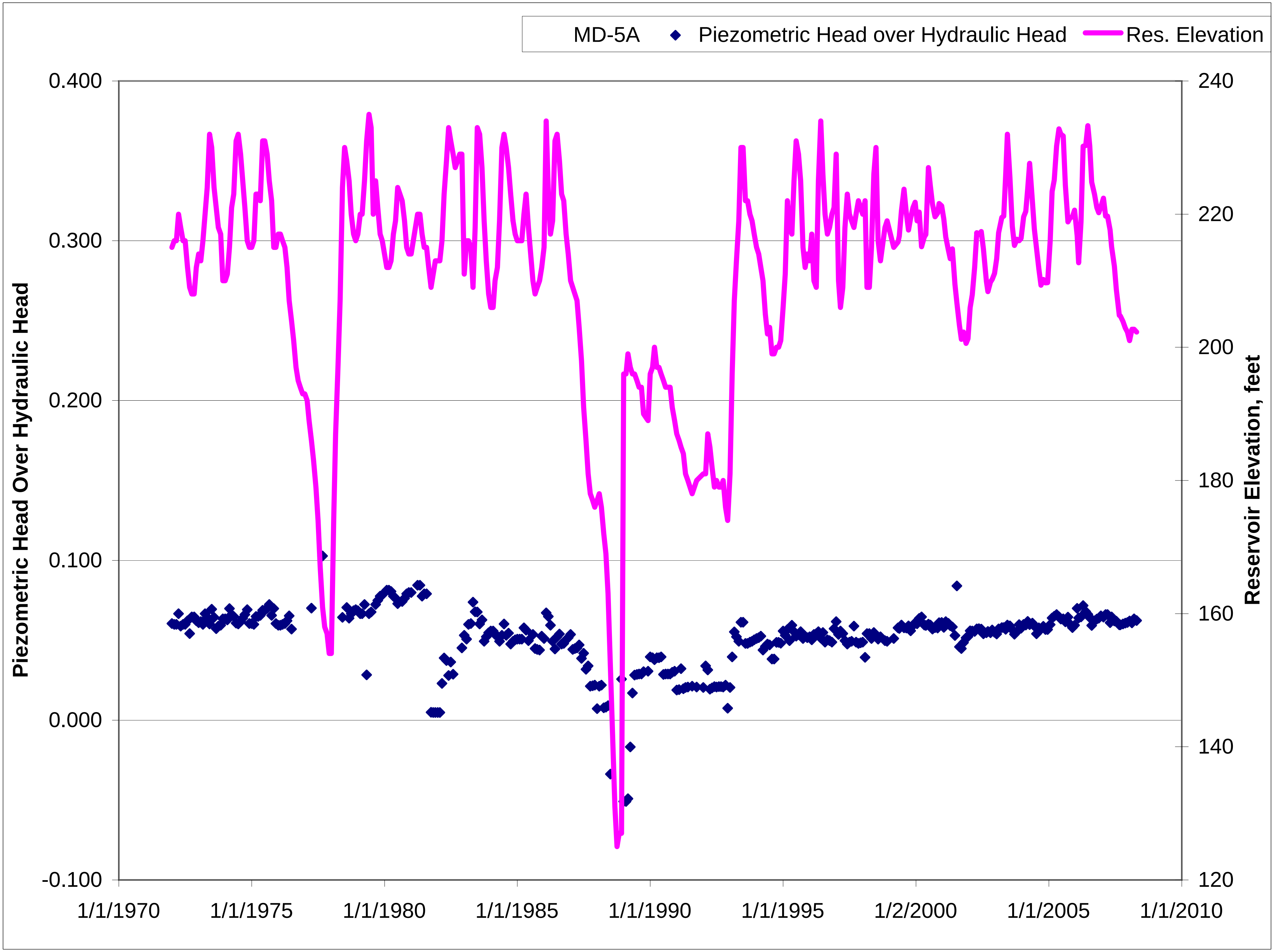

MD-2B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

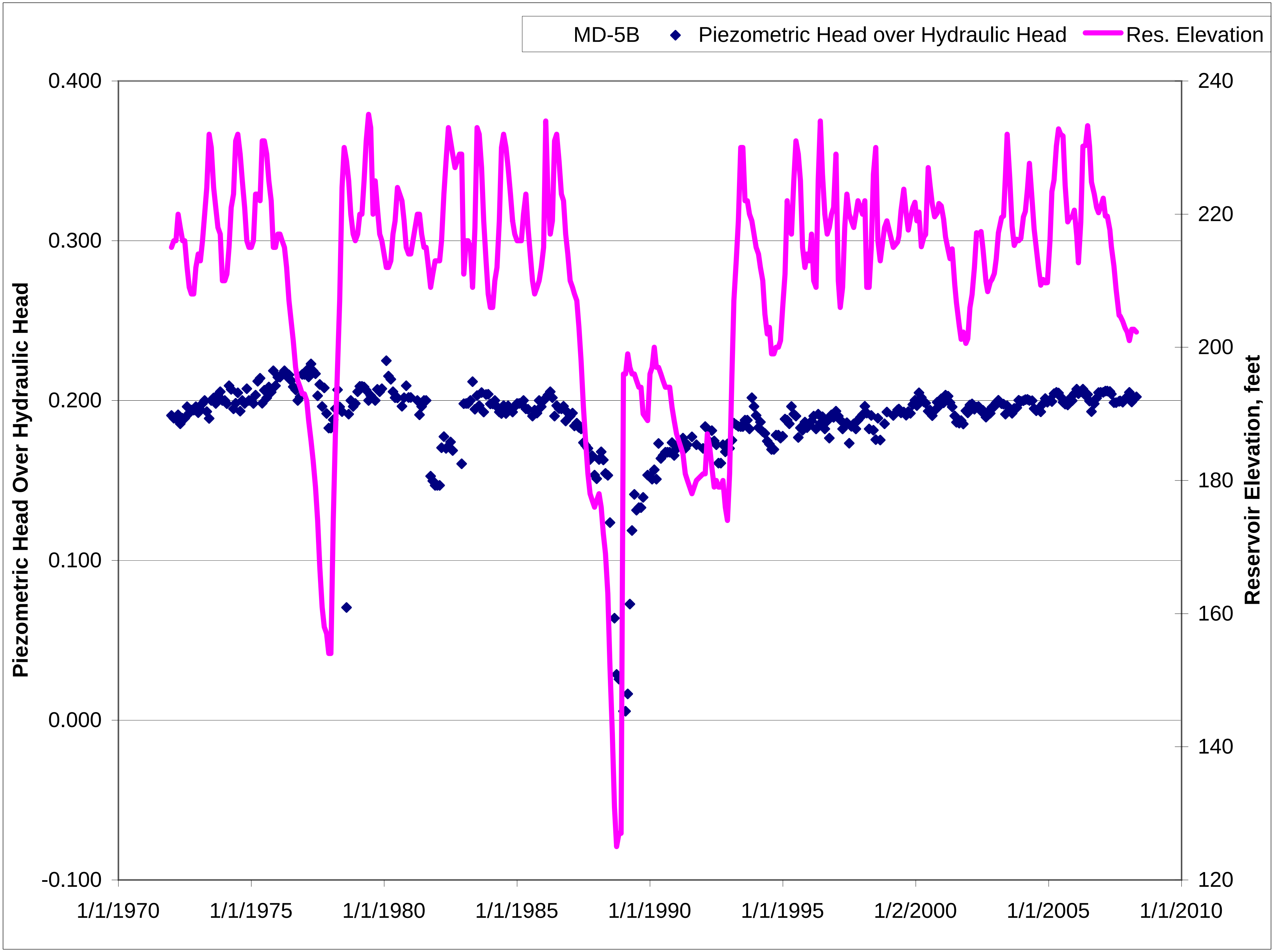

MD-3A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

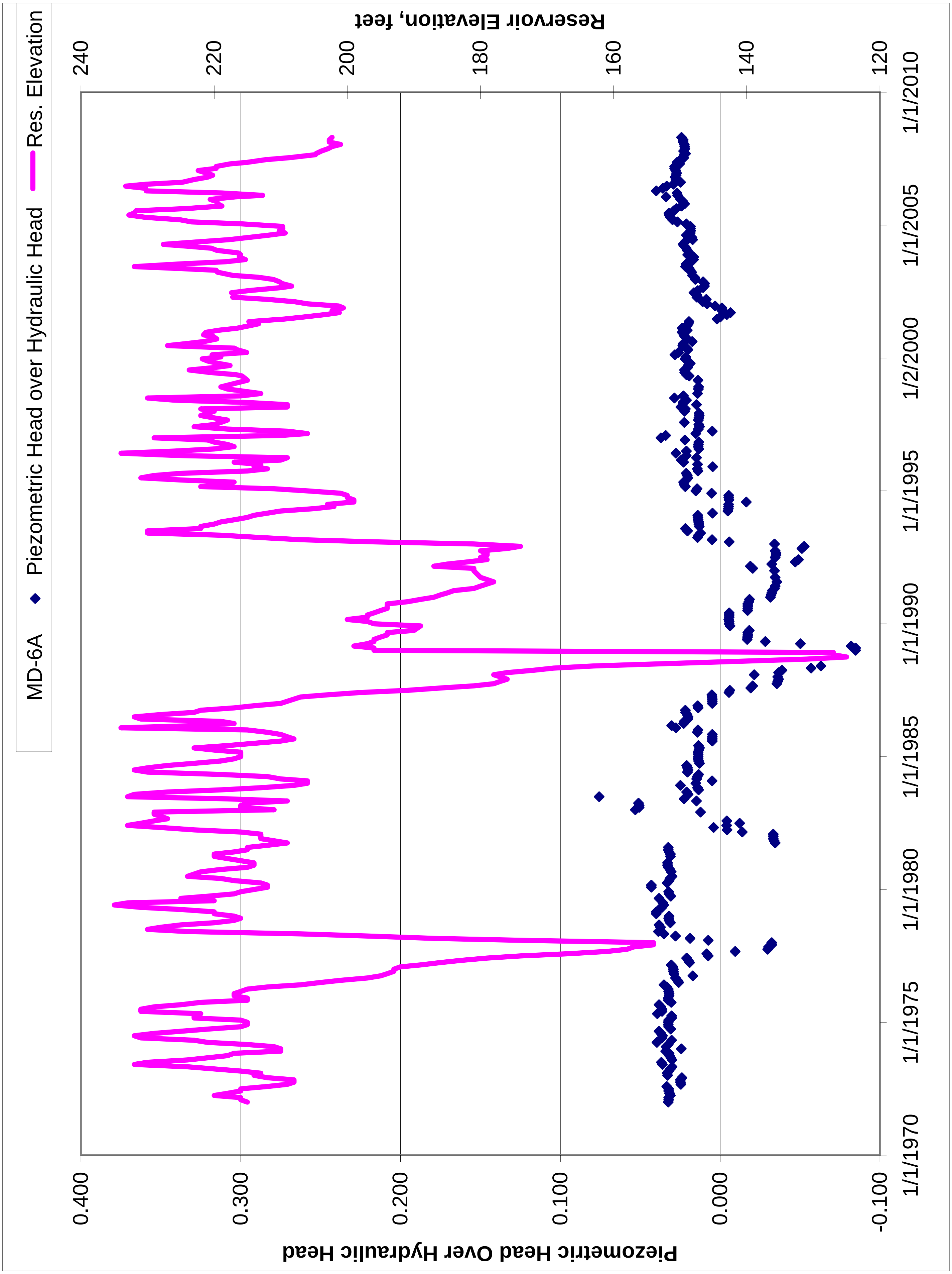

MD-3B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

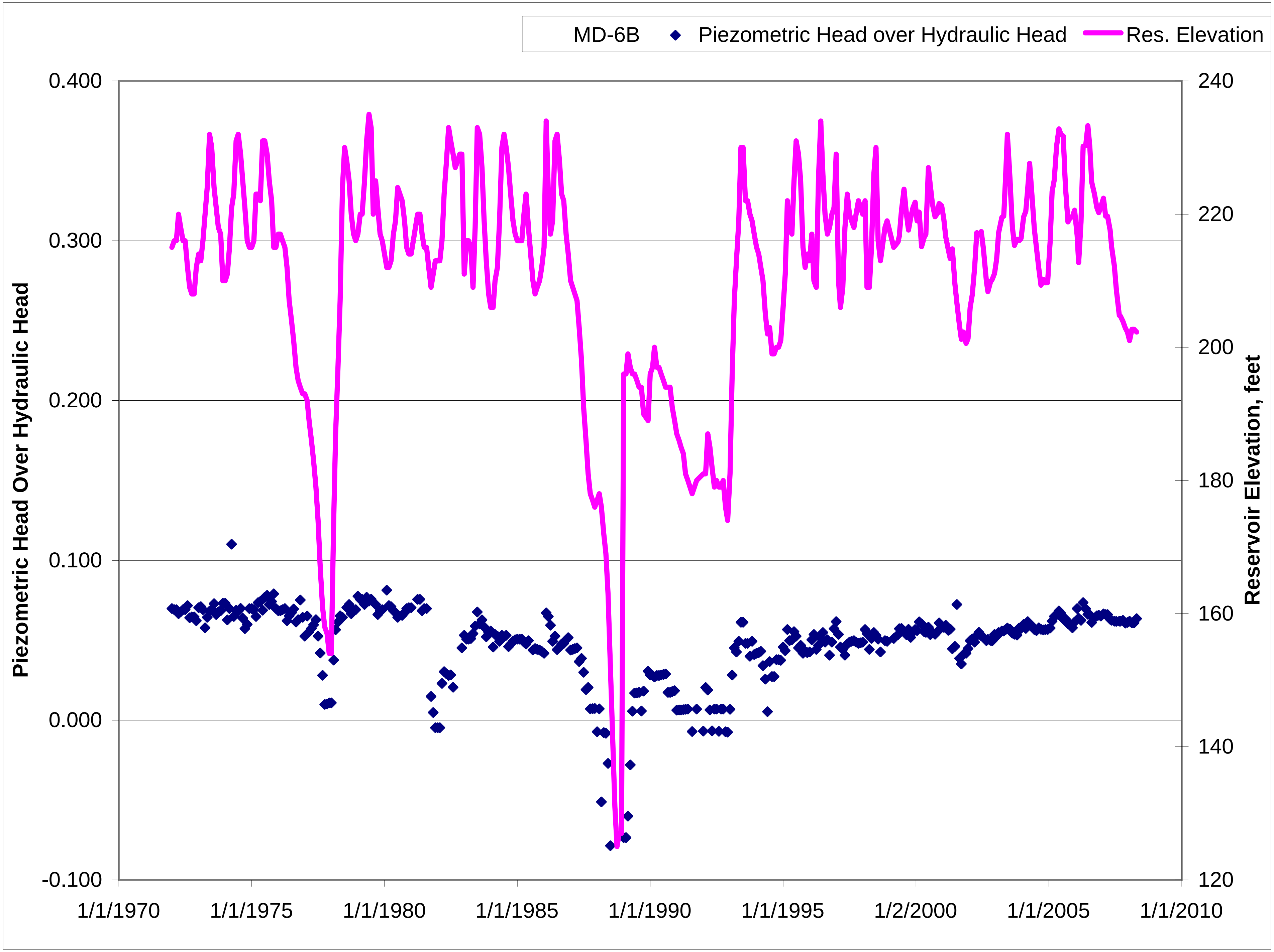

MD-4A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

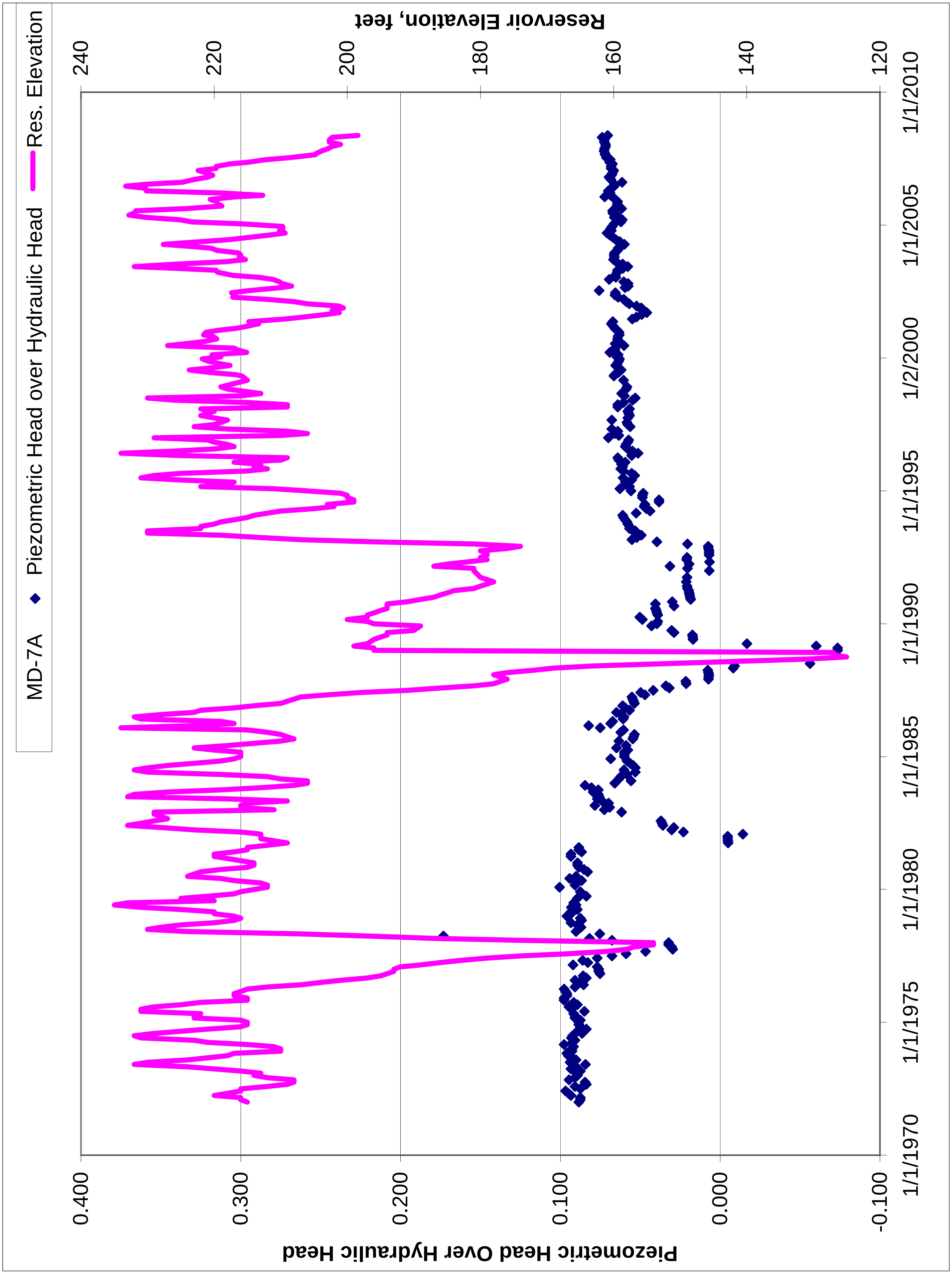

MD-4B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

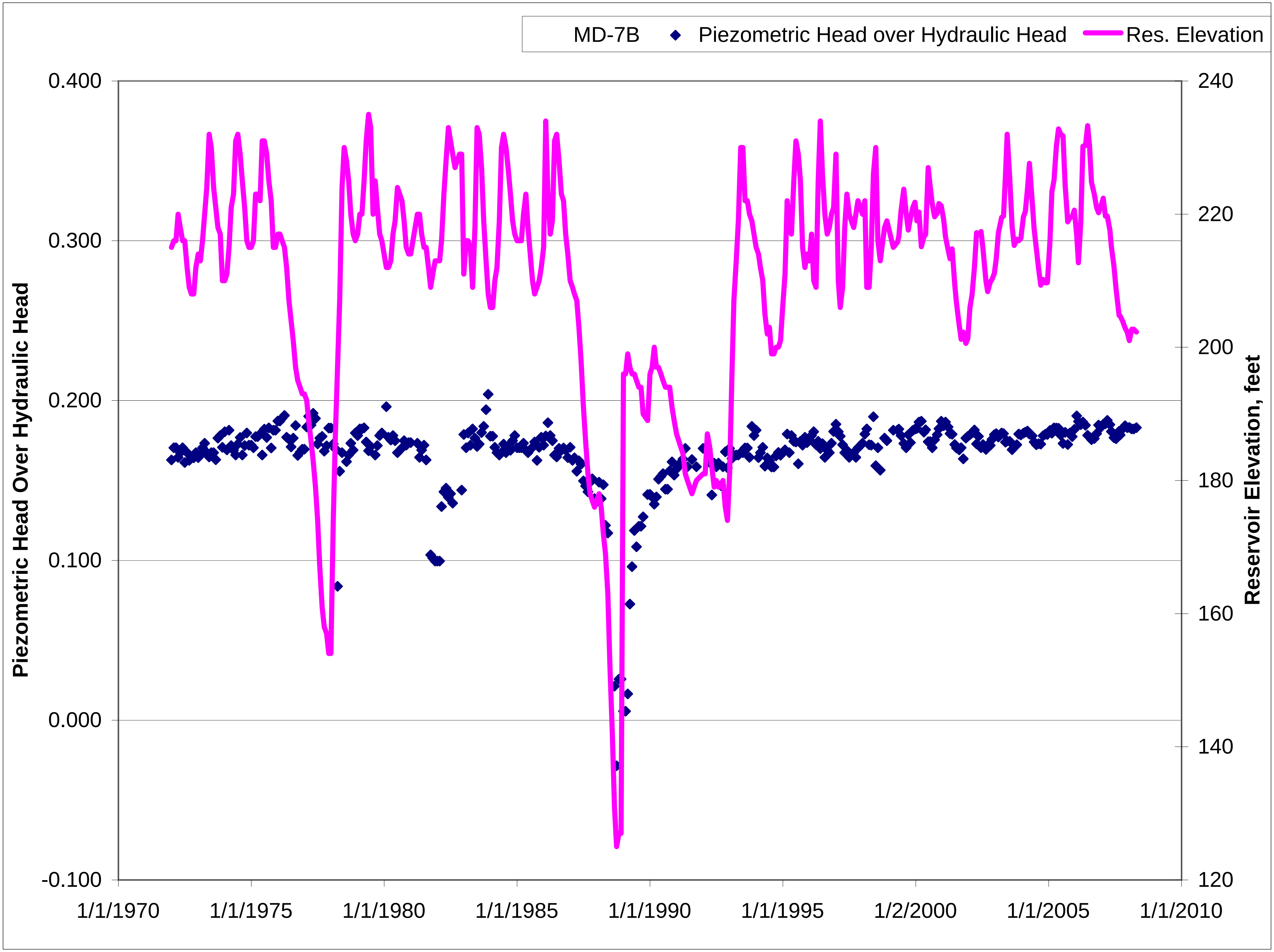

MD-5A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

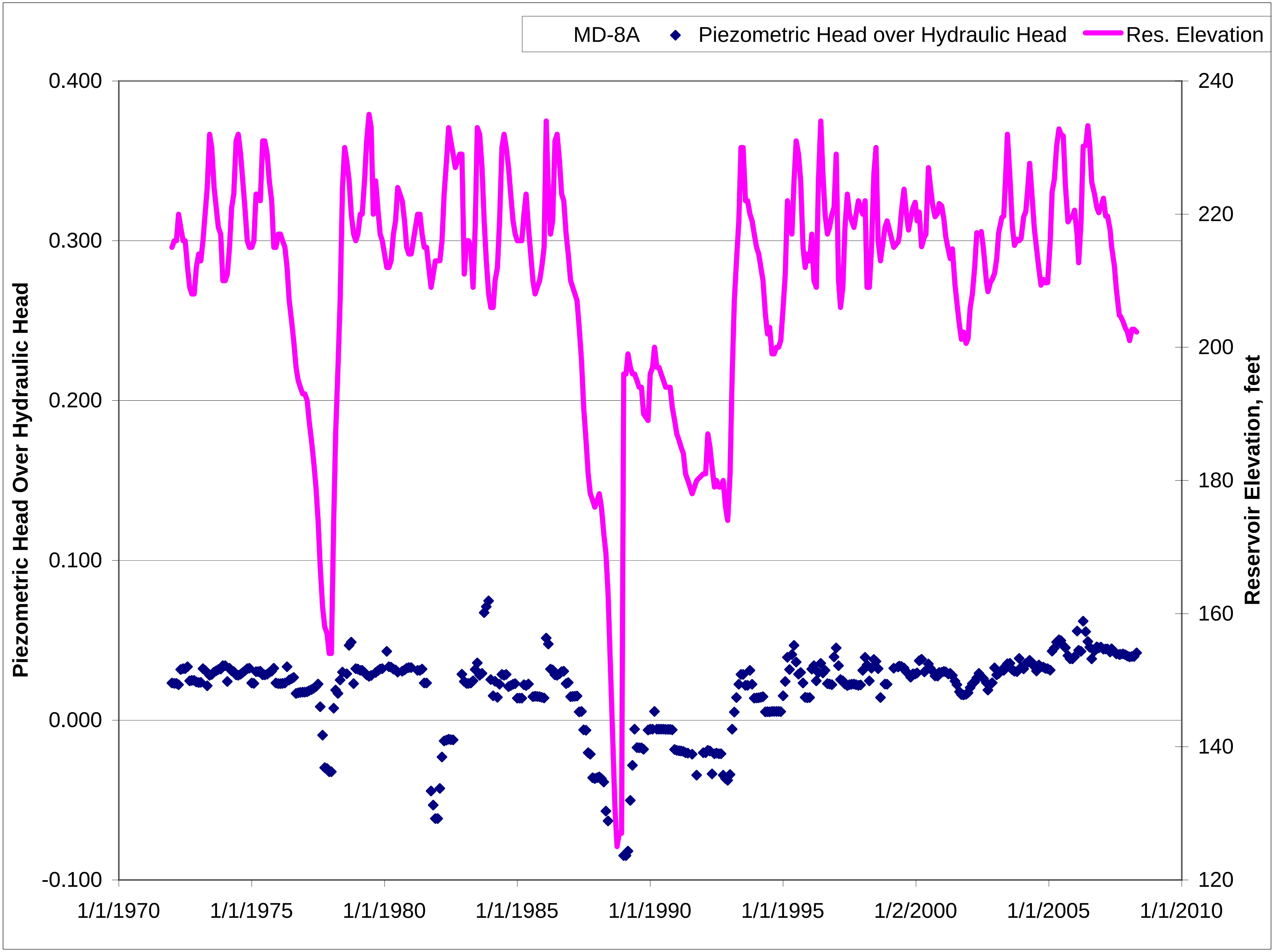

MD-5B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

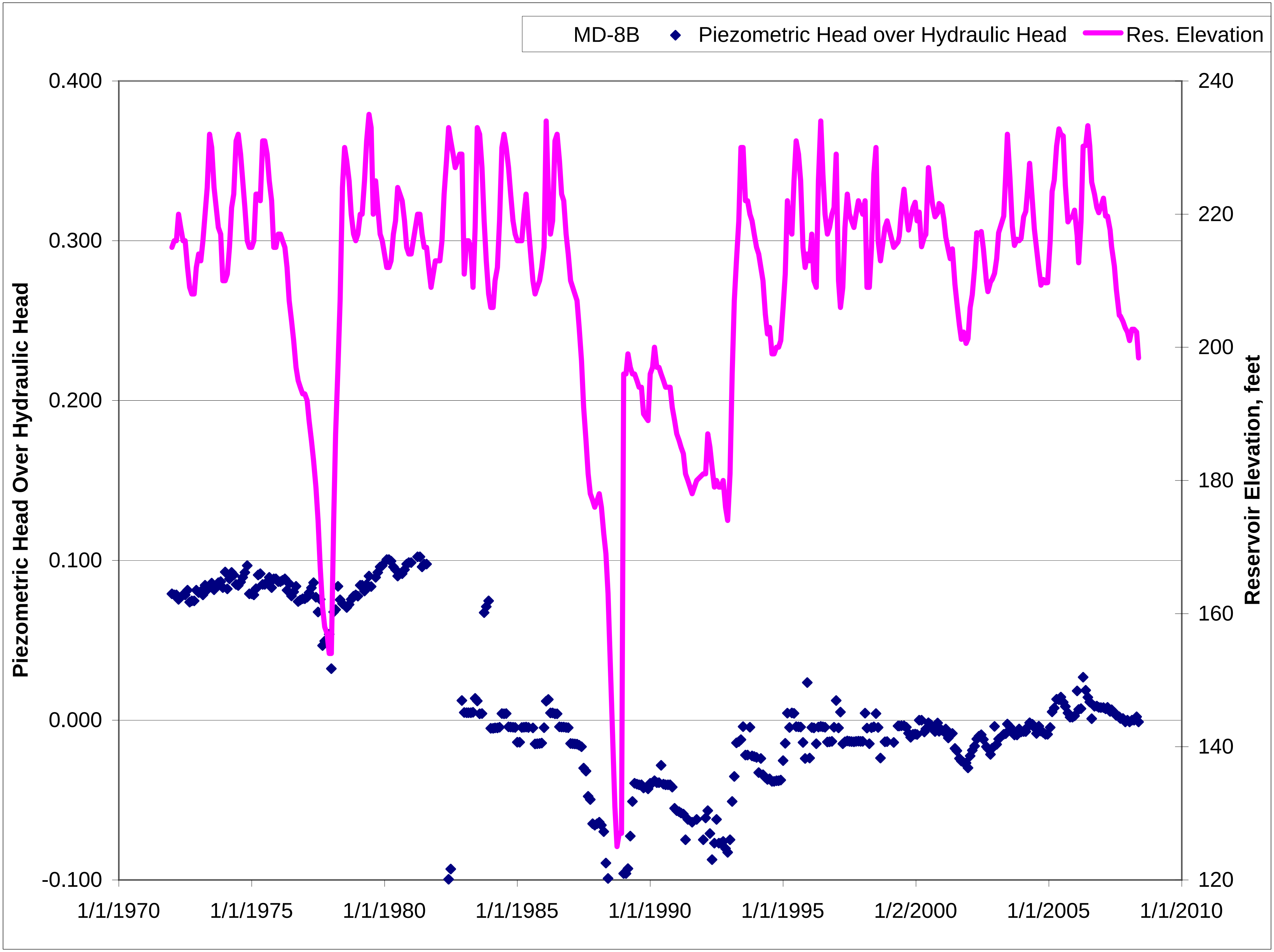

MD-6A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
-0.100
0.000
0.100
0.200
0.300
0.400
120
140
160
180
200
220
240
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

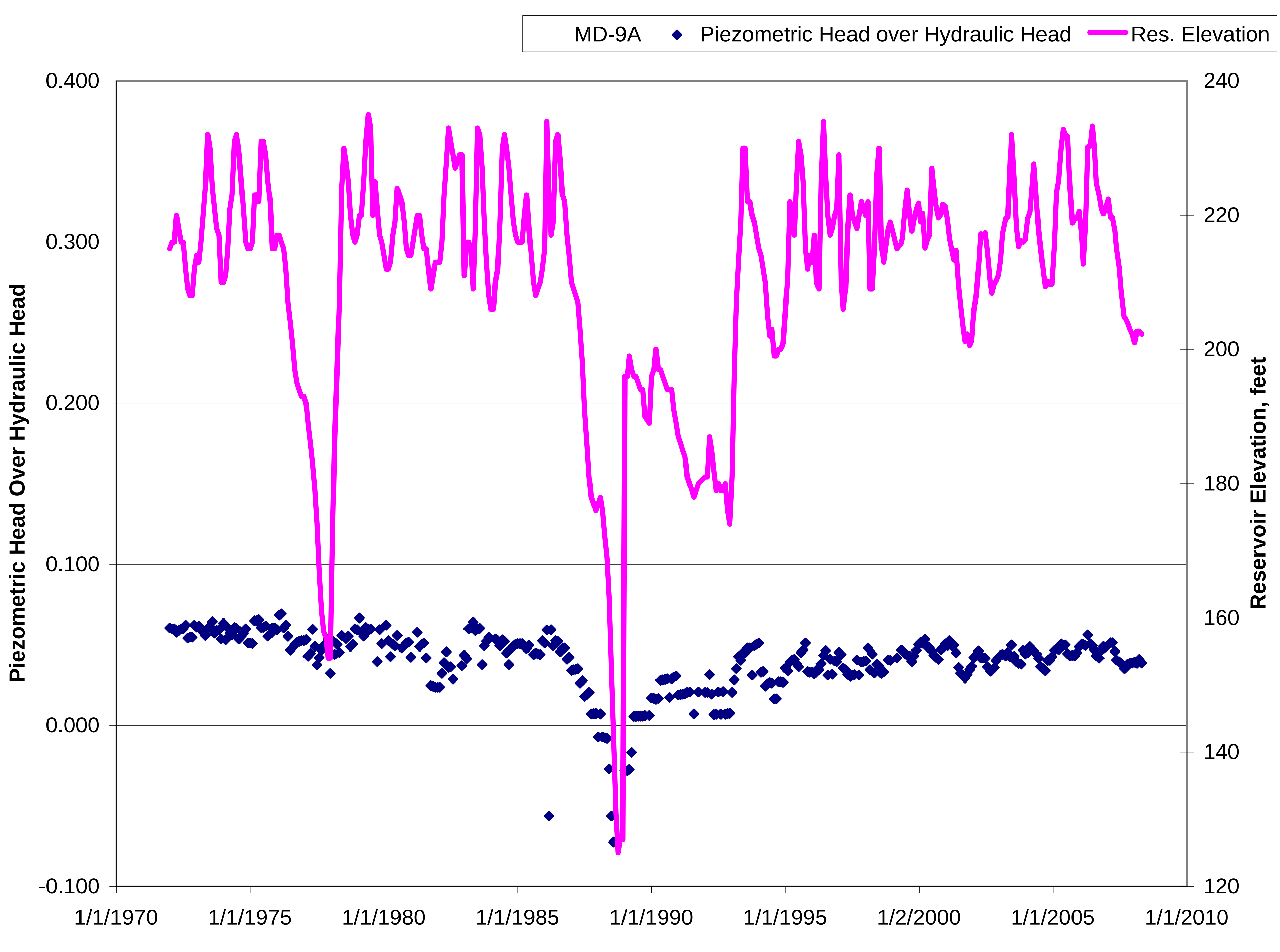

MD-6B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010


MD-7A
Piezometric Head over Hydraulic Head
Res. Elevation
Reservoir Elevation, feet
Piezometric Head Over Hydraulic Head
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

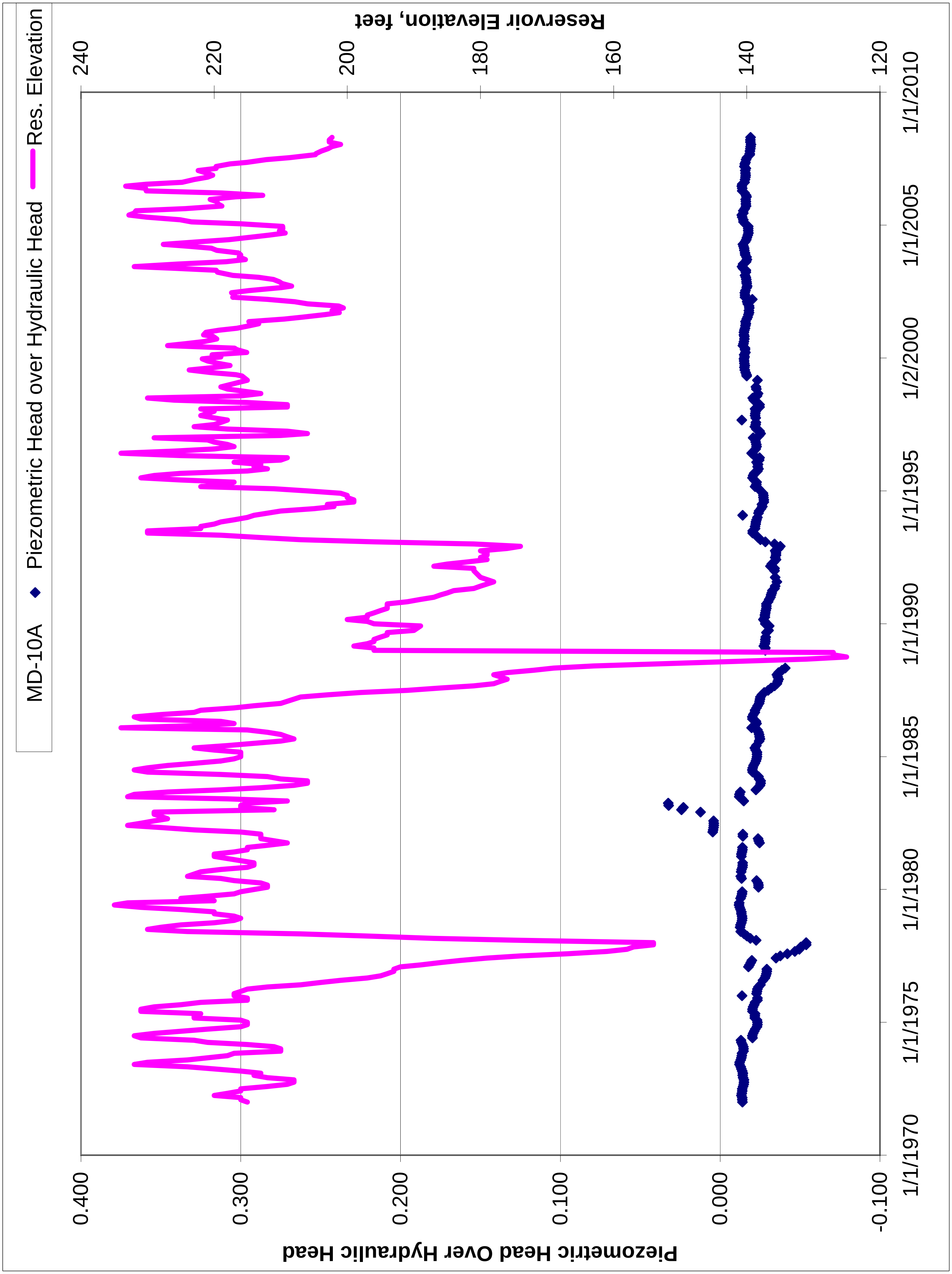

MD-7B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

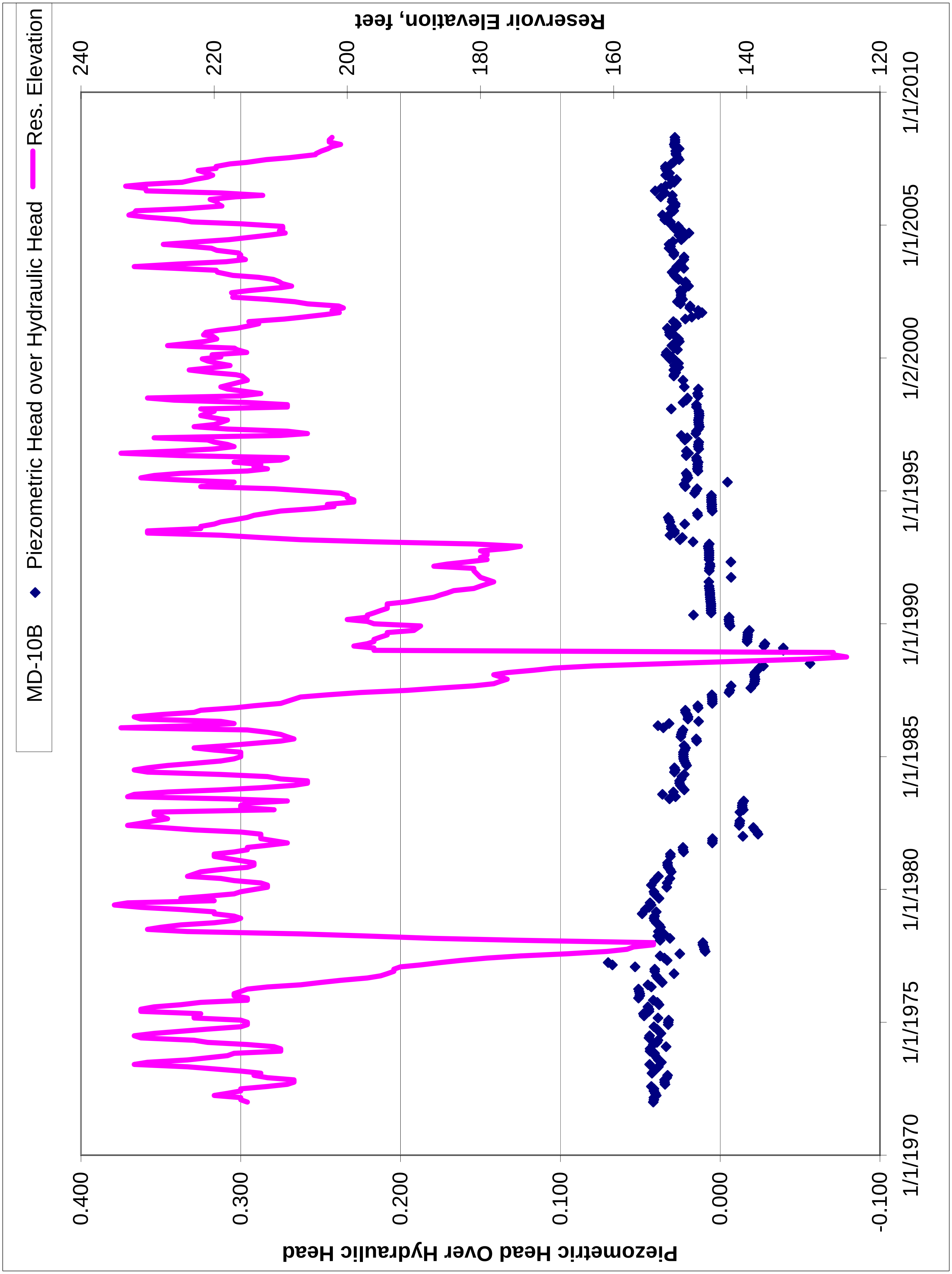

MD-8A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

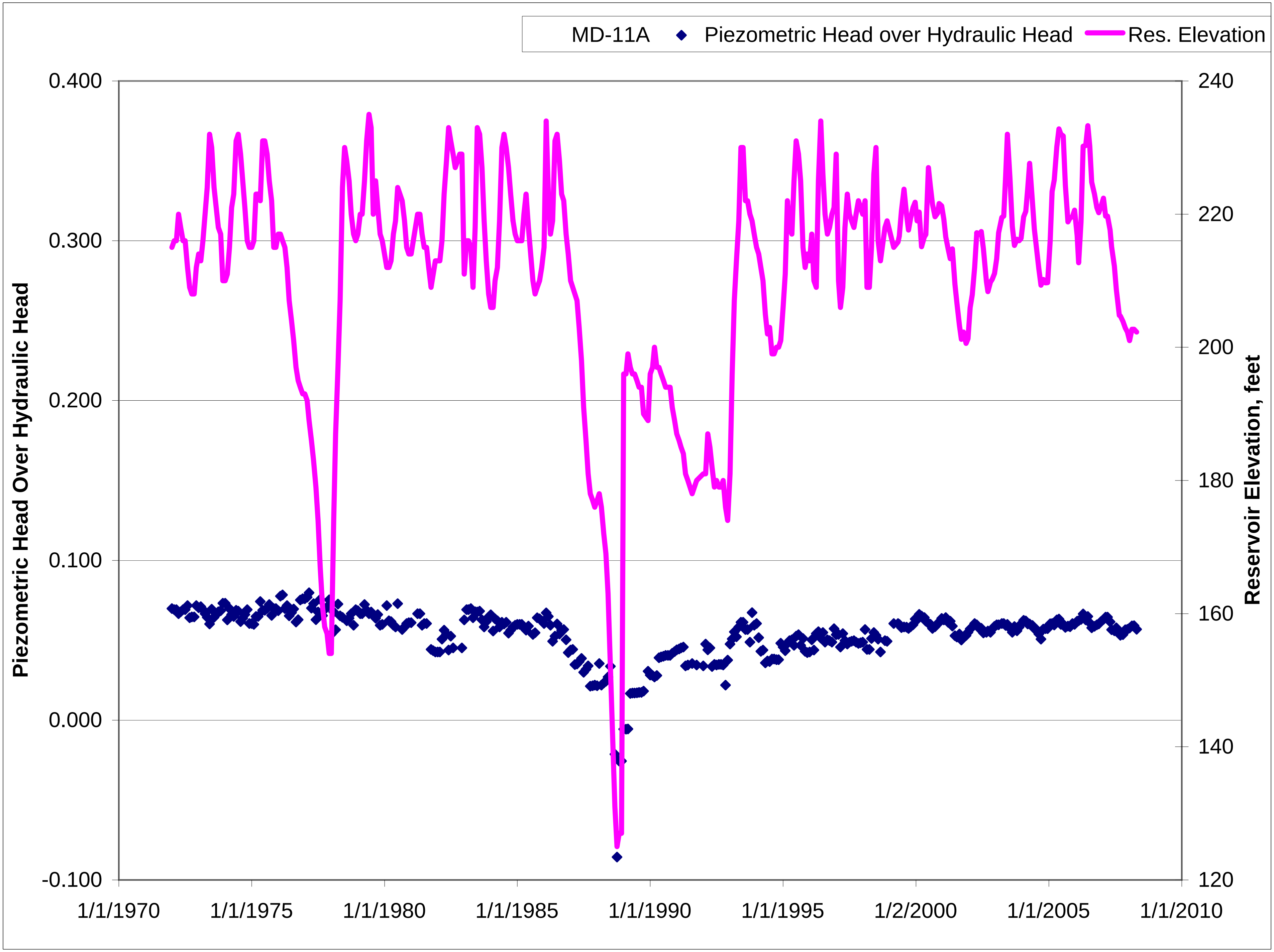

MD-8B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

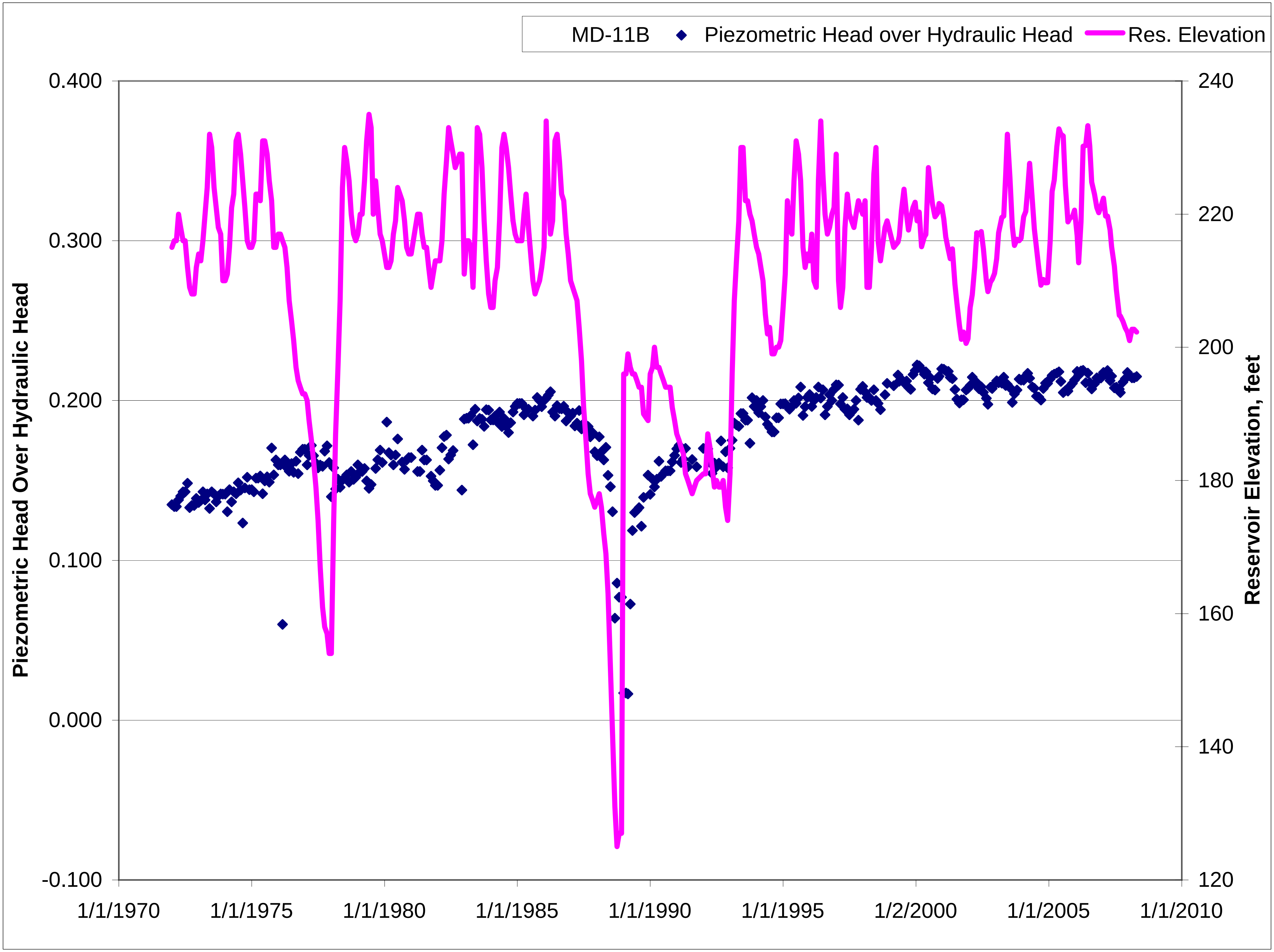


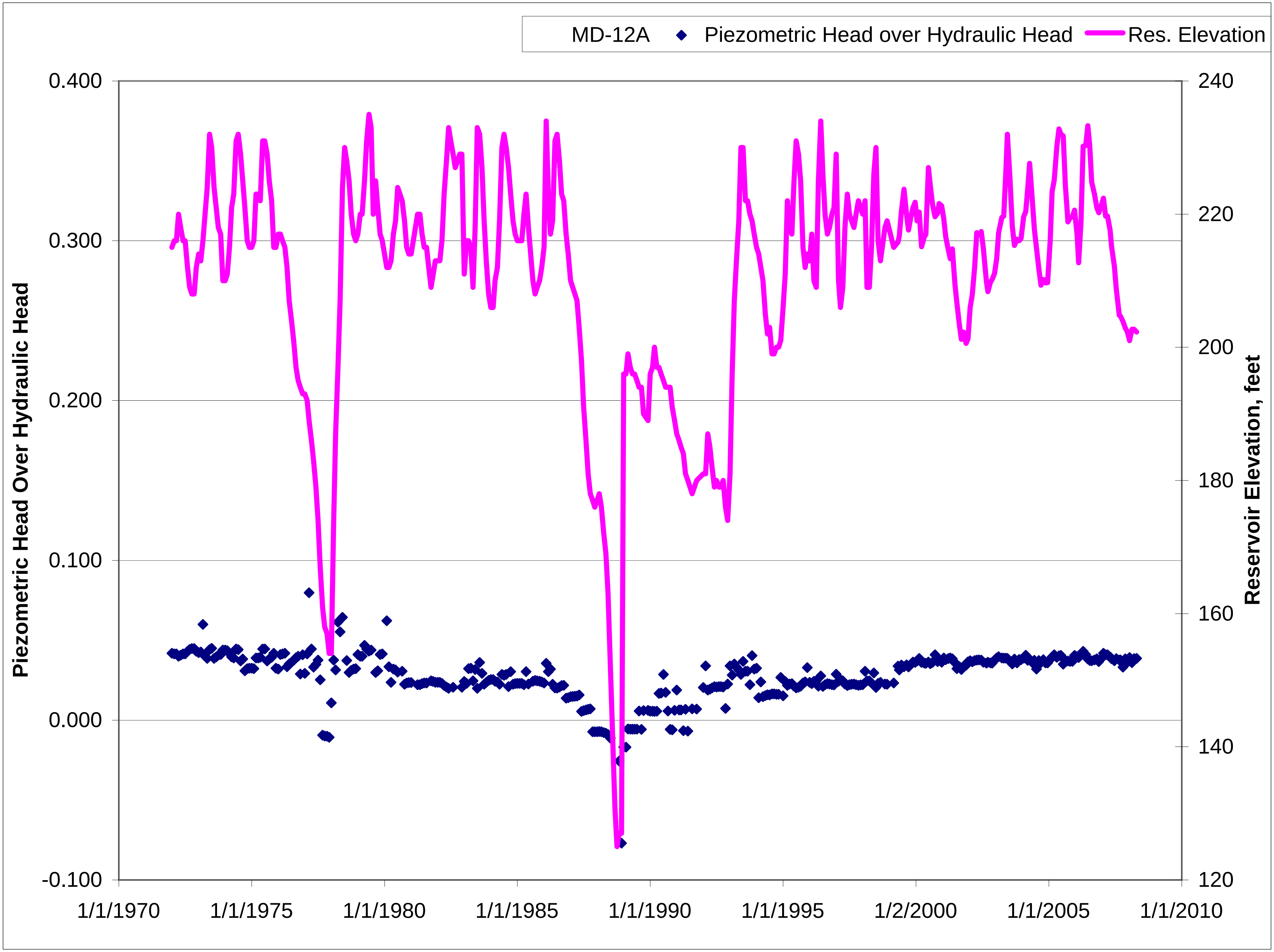
MD-9A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010


MD-10A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
-0.100
0.000
0.100
0.200
0.300
0.400
120
140
160
180
200
220
240
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

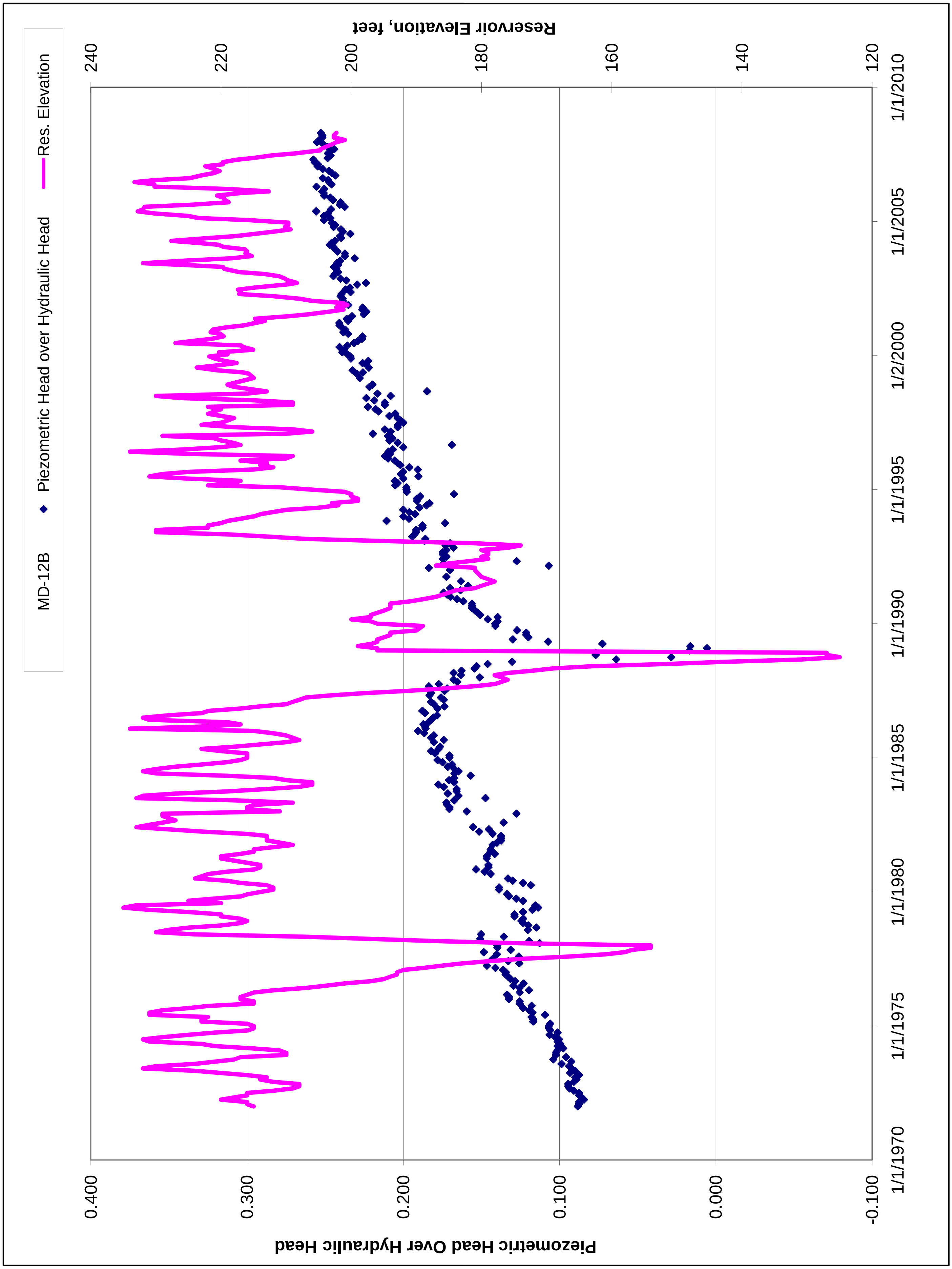

MD-10B
Piezometric Head over Hydraulic Head
Res. Elevation
Reservoir Elevation, feet
Piezometric Head Over Hydraulic Head
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010


MD-11A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

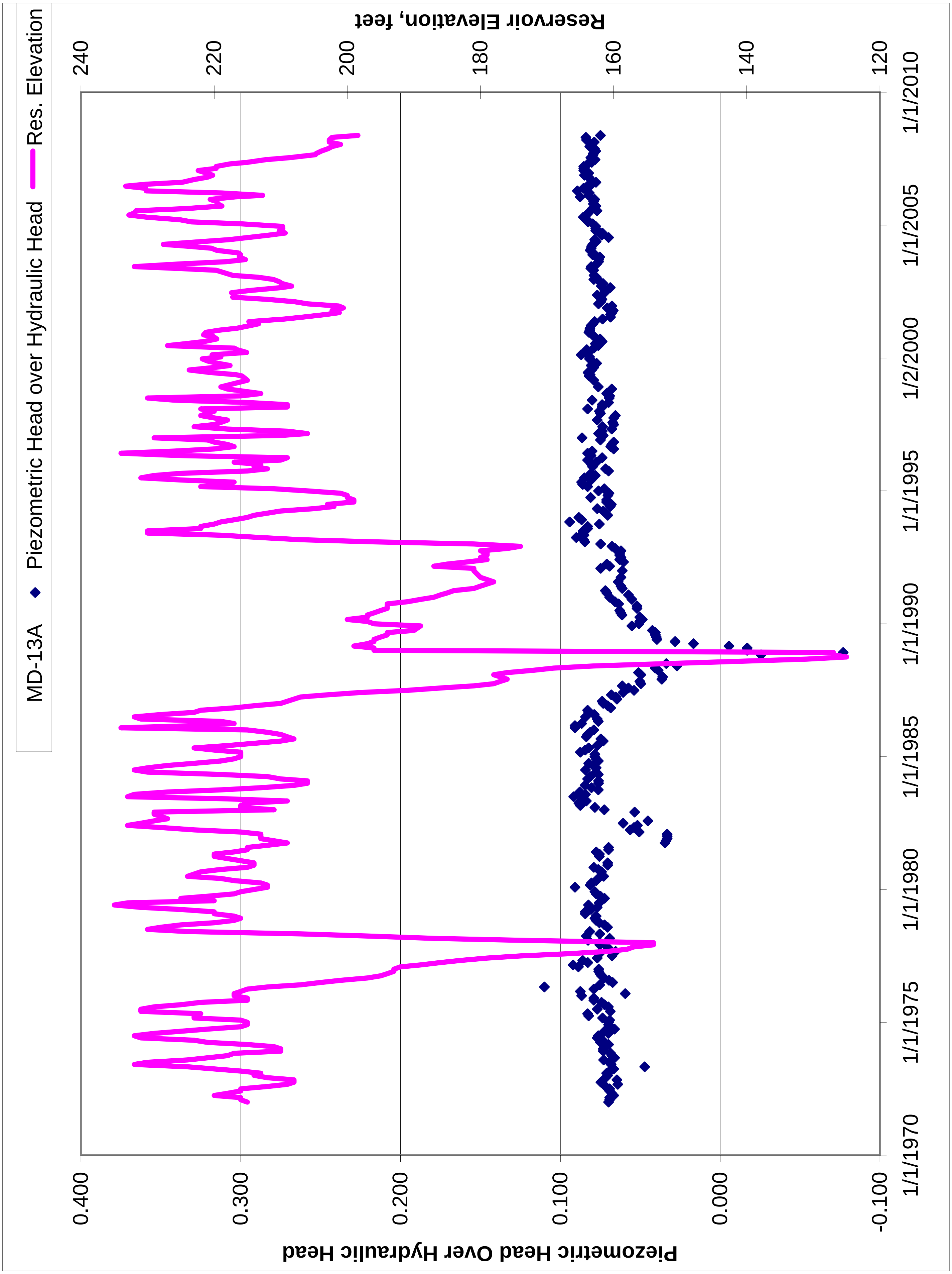

MD-11B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010



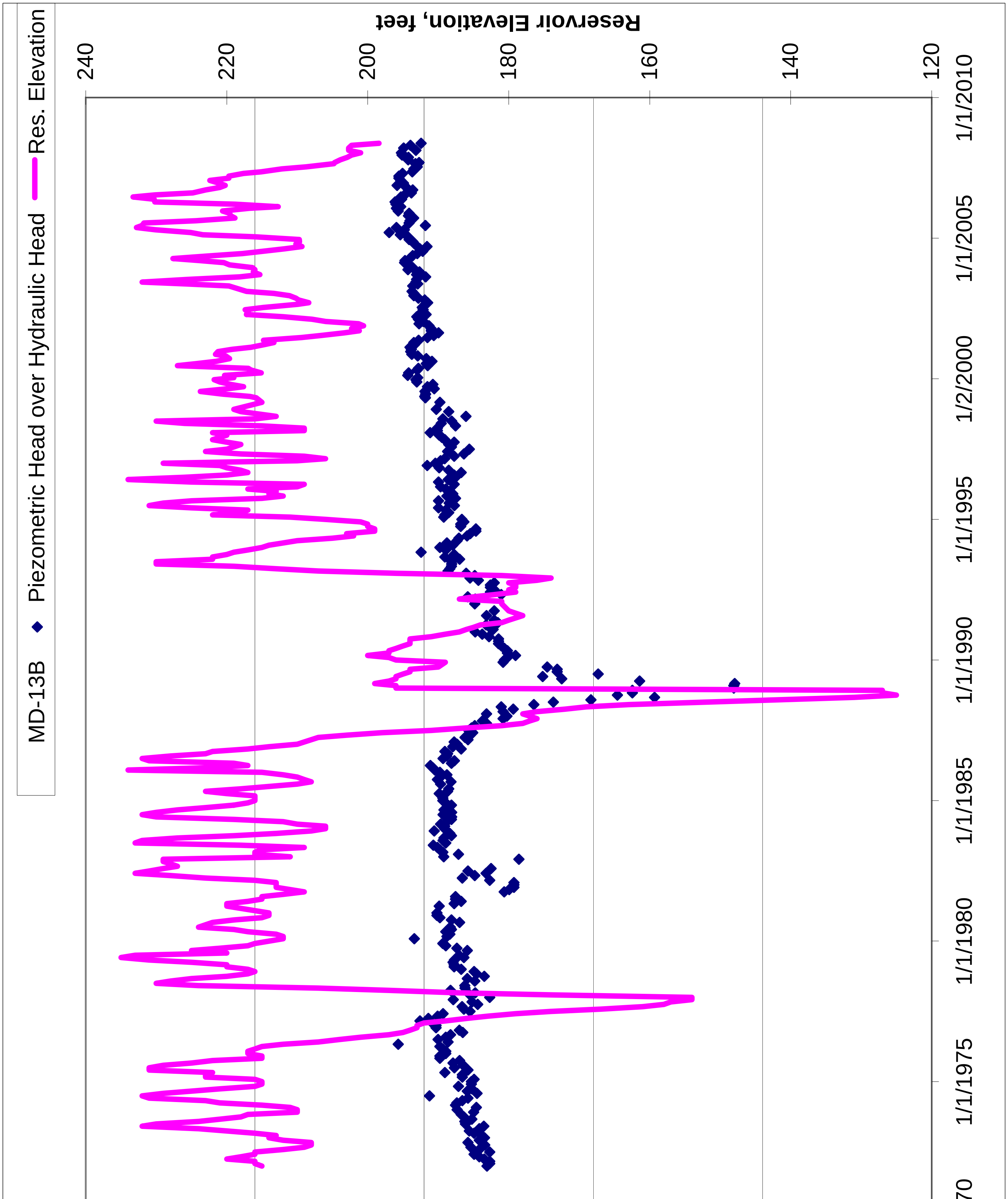
MD-12A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

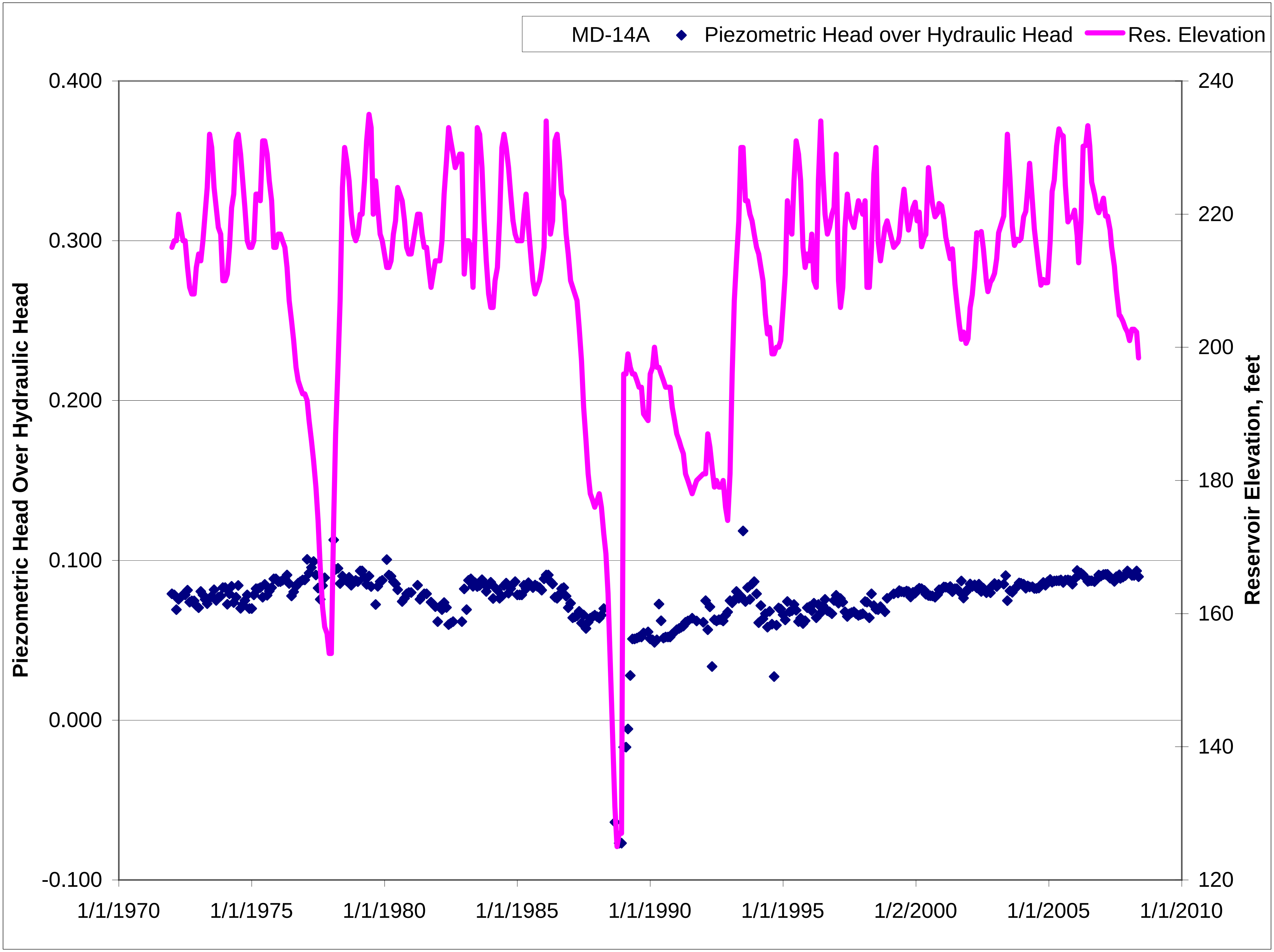


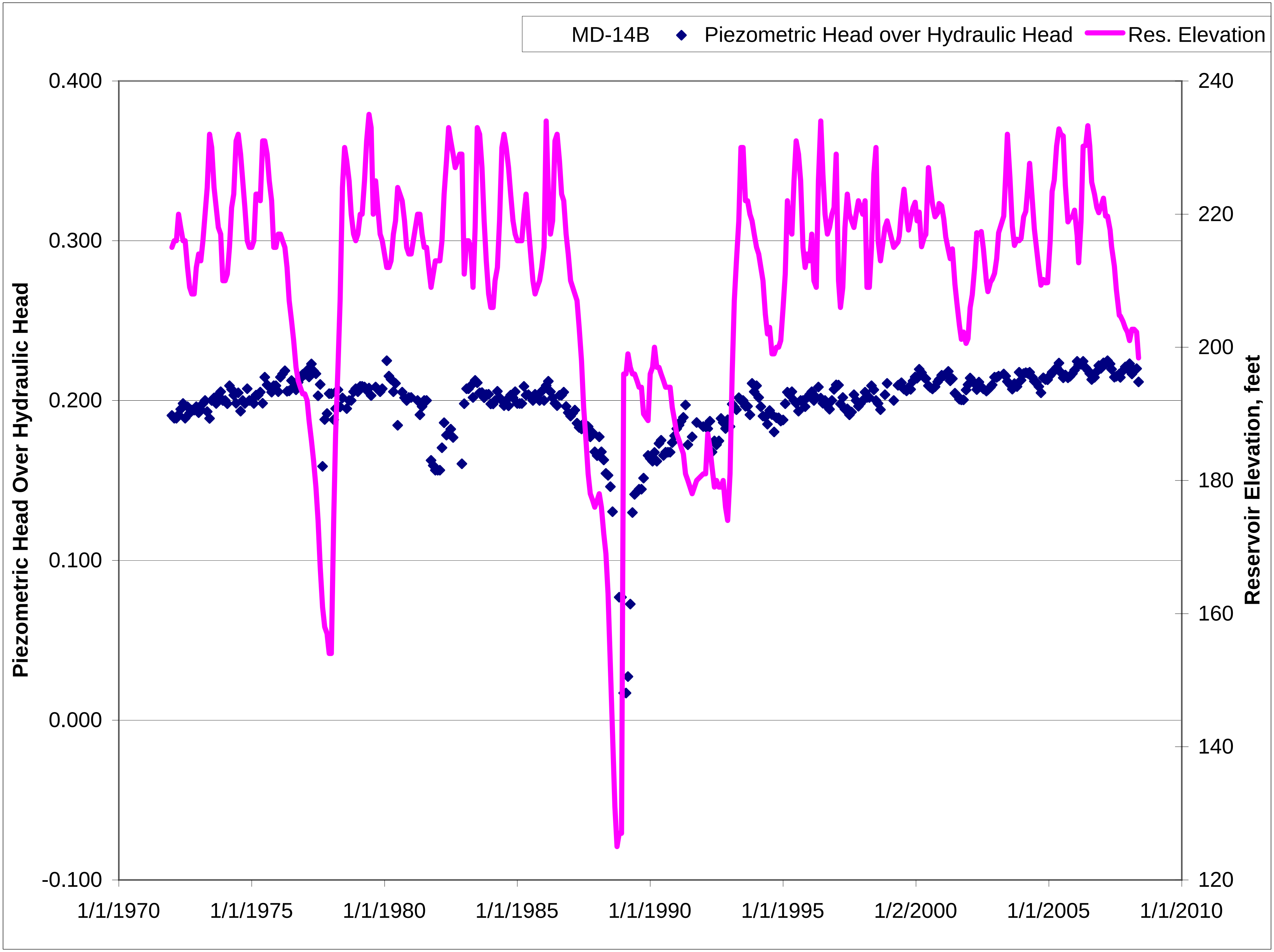
MD-12B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
-0.100
0.000
0.100
0.200
0.300
0.400
120
140
160
180
200
220
240
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

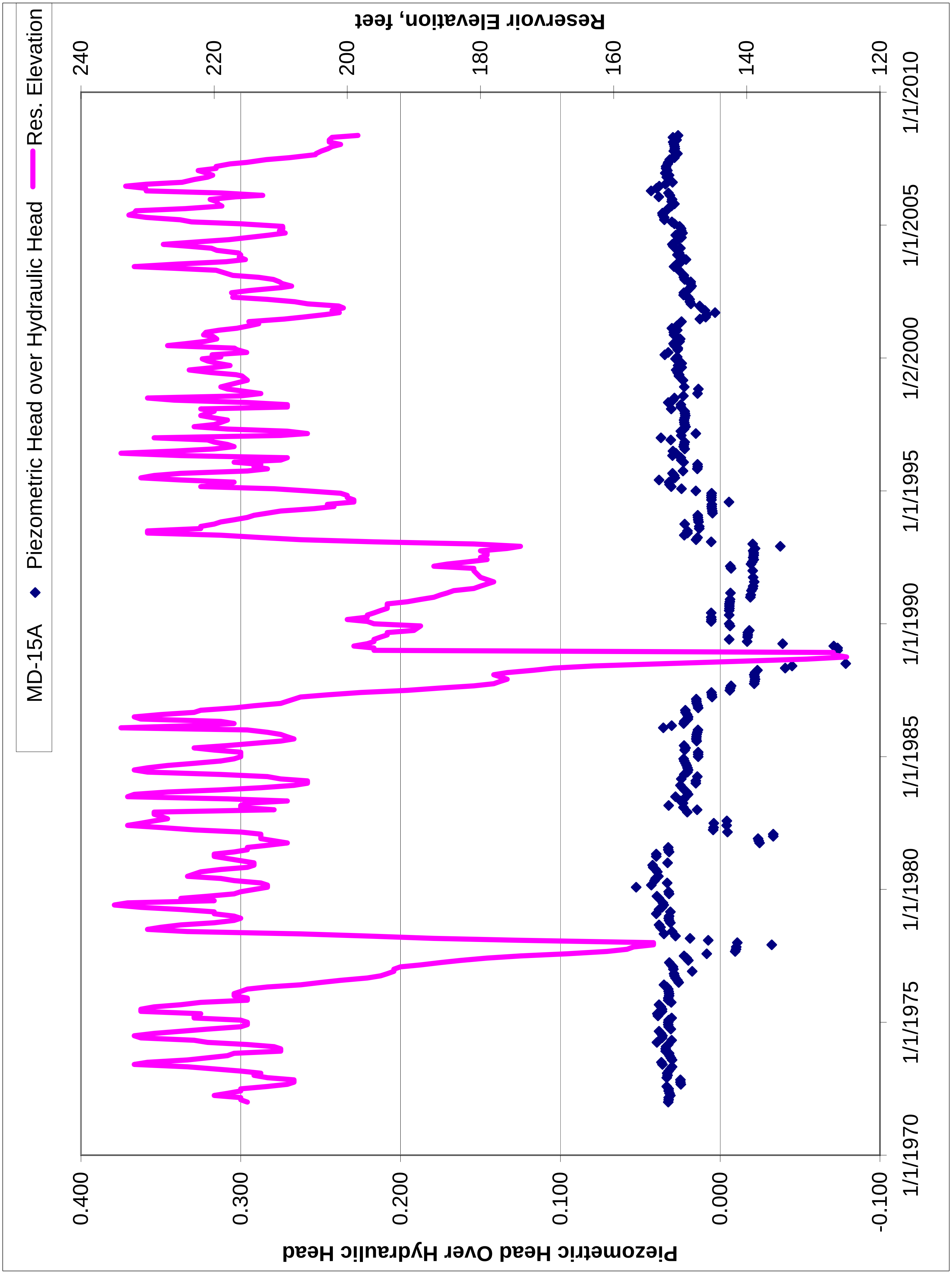


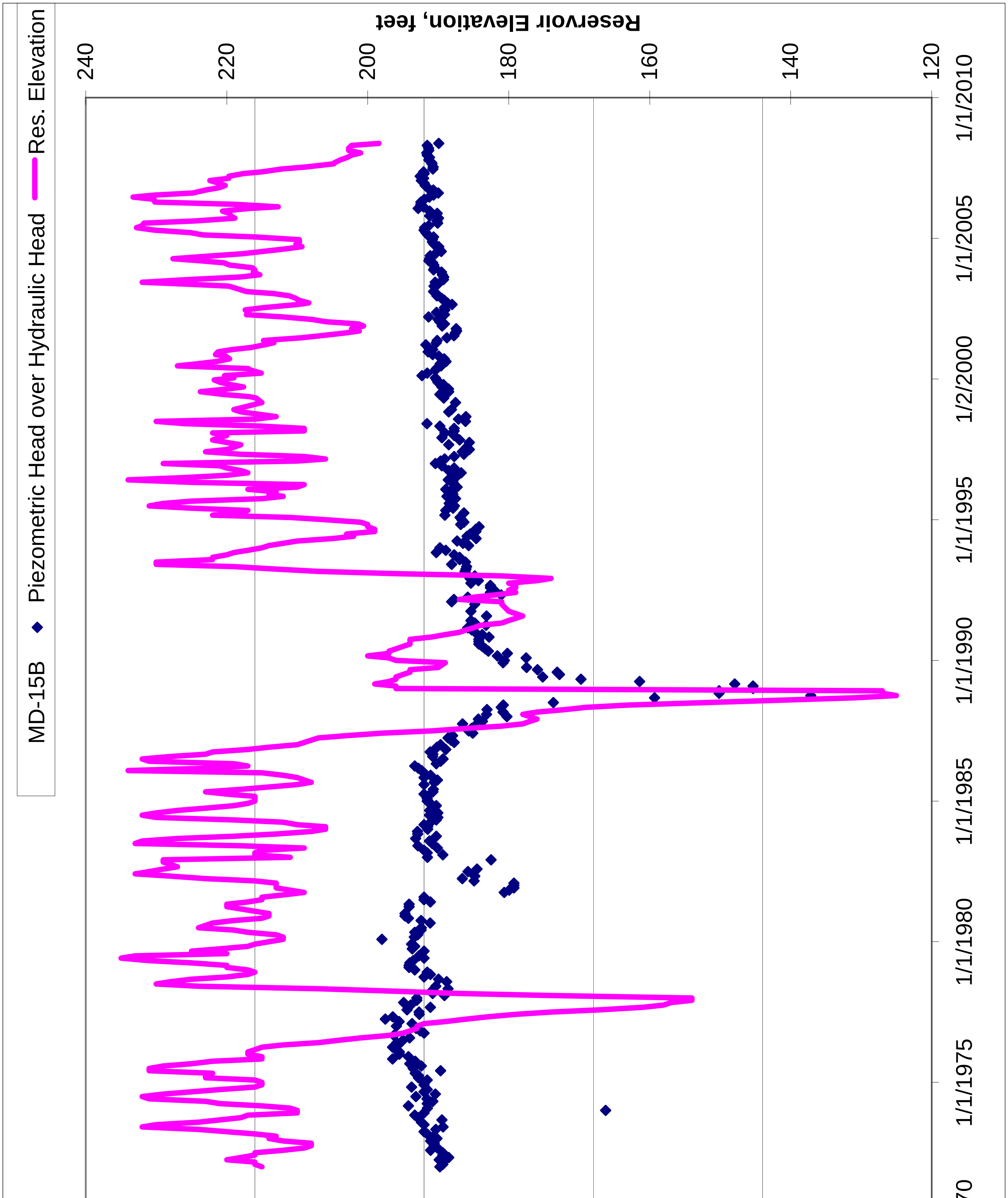
MD-13A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

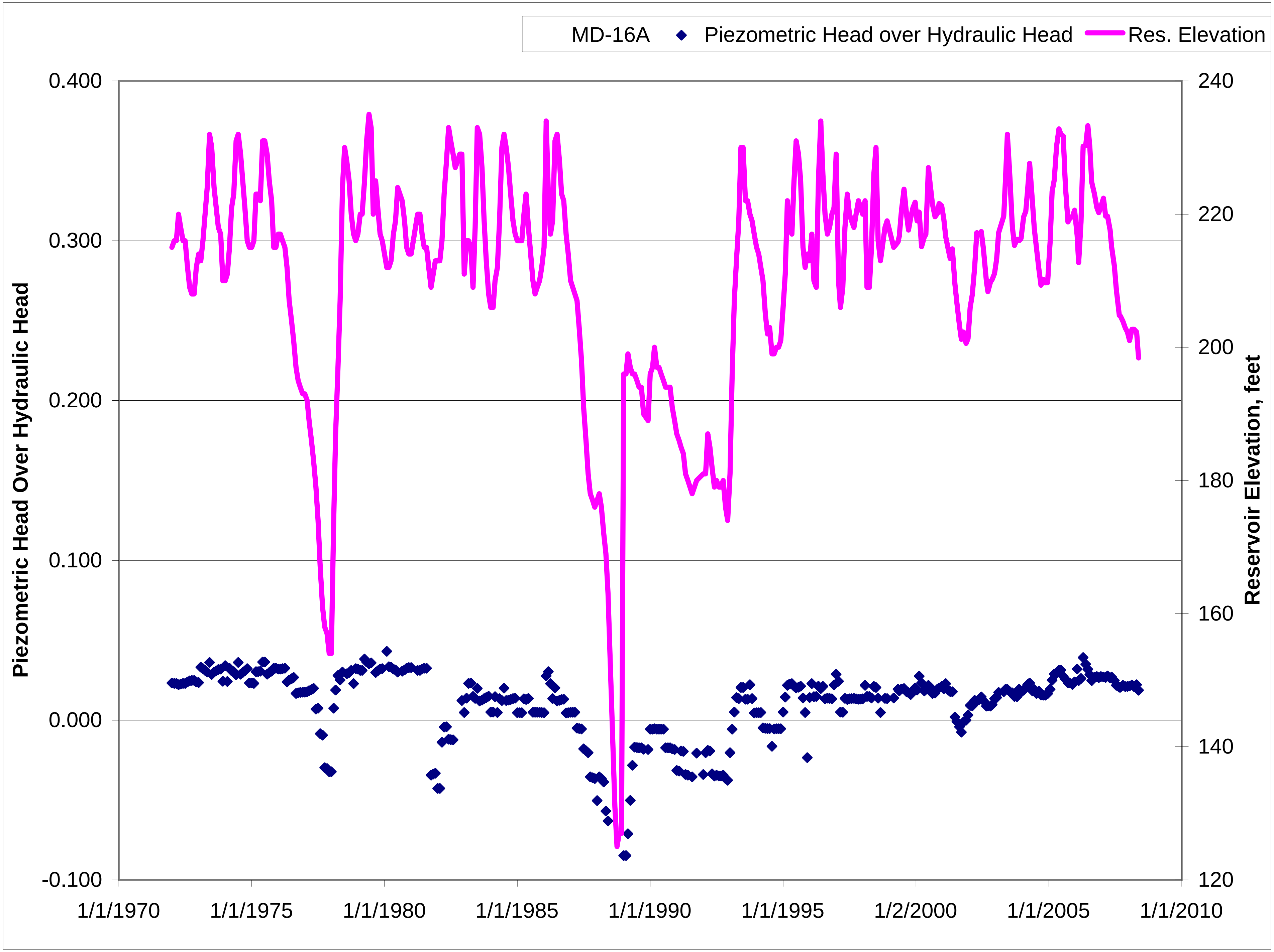

MD-13B
Piezometric Head over Hydraulic Head
Res. Elevation
Reservoir Elevation, feet
Piezometric Head Over Hydraulic Head
240
220
200
180
160
140
120
0.400
0.300
0.200
0.100
0.000
-0.100
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

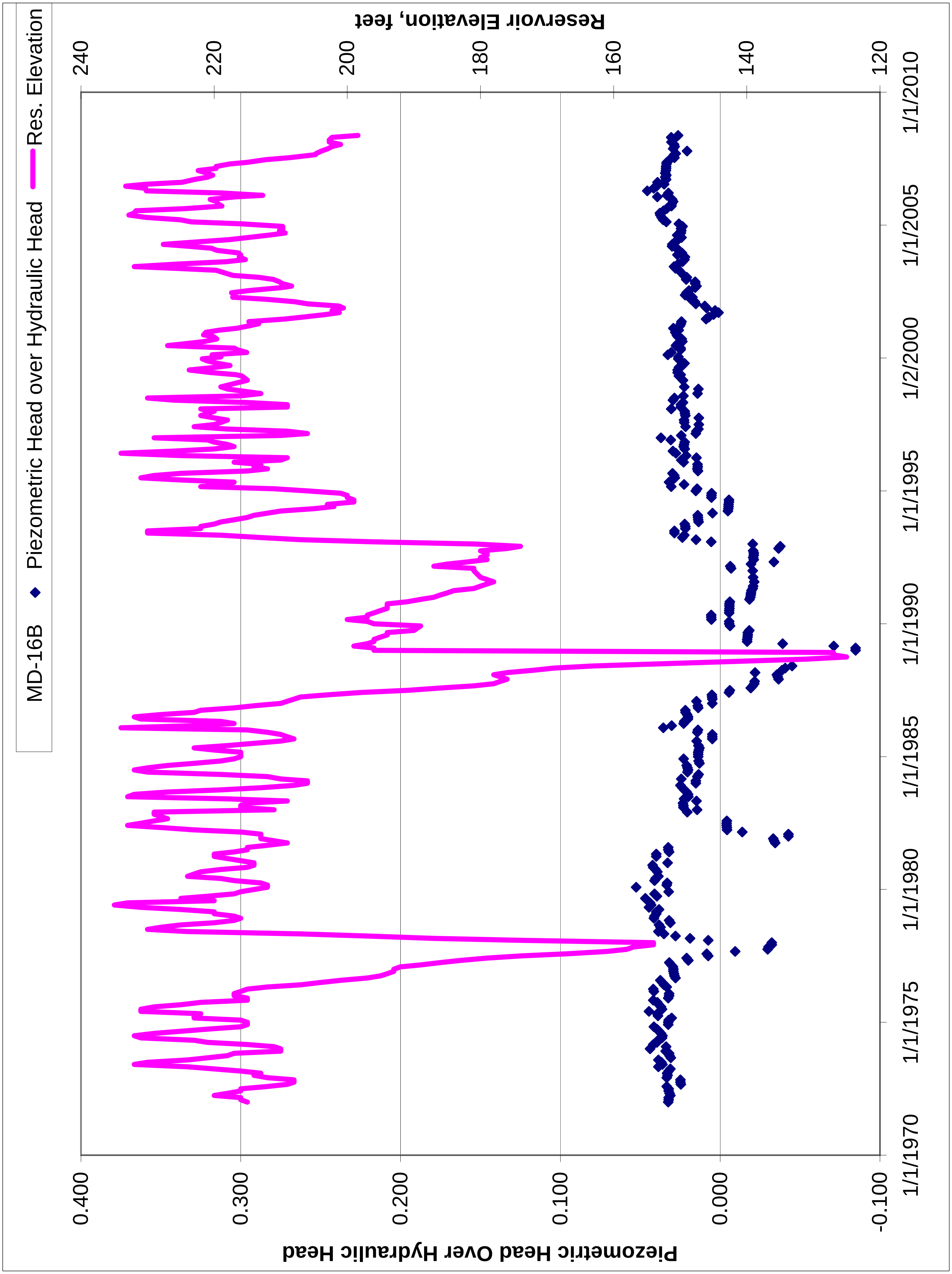

MD-14A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

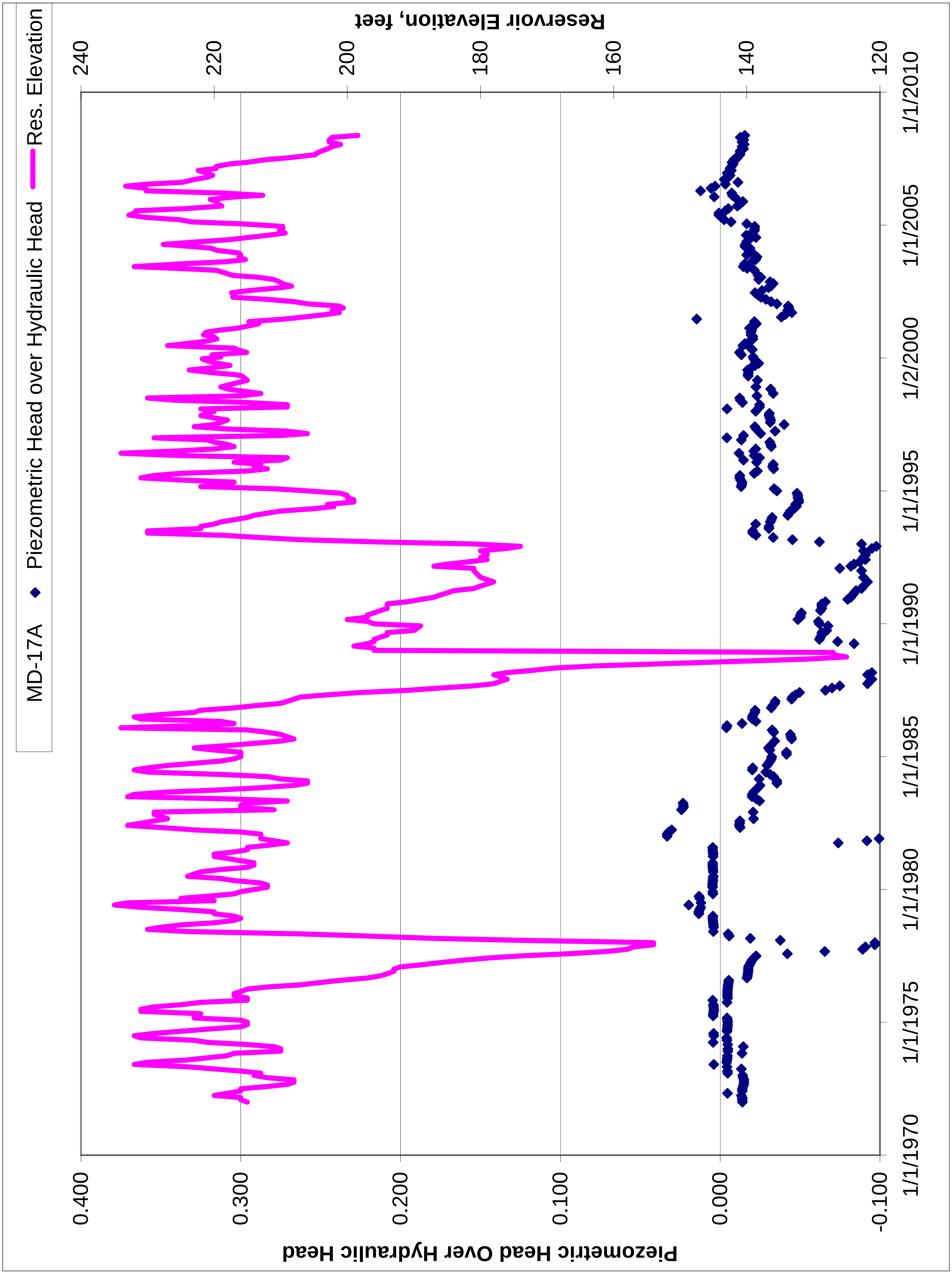

MD-14B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

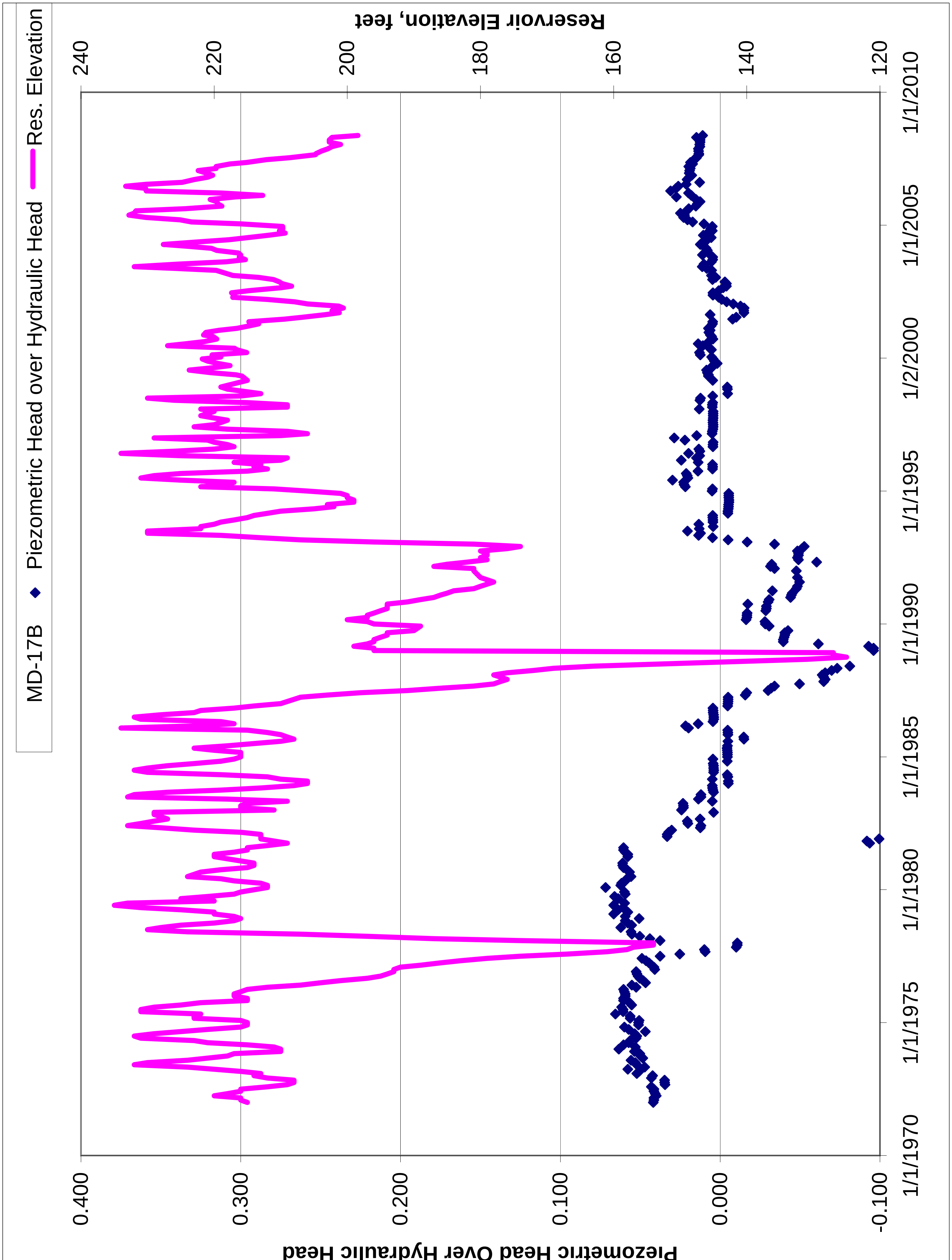

MD-15A
Piezometric Head over Hydraulic Head
Res. Elevation
Reservoir Elevation, feet
240
220
200
180
160
140
120
Piezometric Head Over Hydraulic Head
0.400
0.300
0.200
0.100
0.000
-0.100
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010


MD-15B
Piezometric Head over Hydraulic Head
Res. Elevation
Reservoir Elevation, feet
Piezometric Head Over Hydraulic Head
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

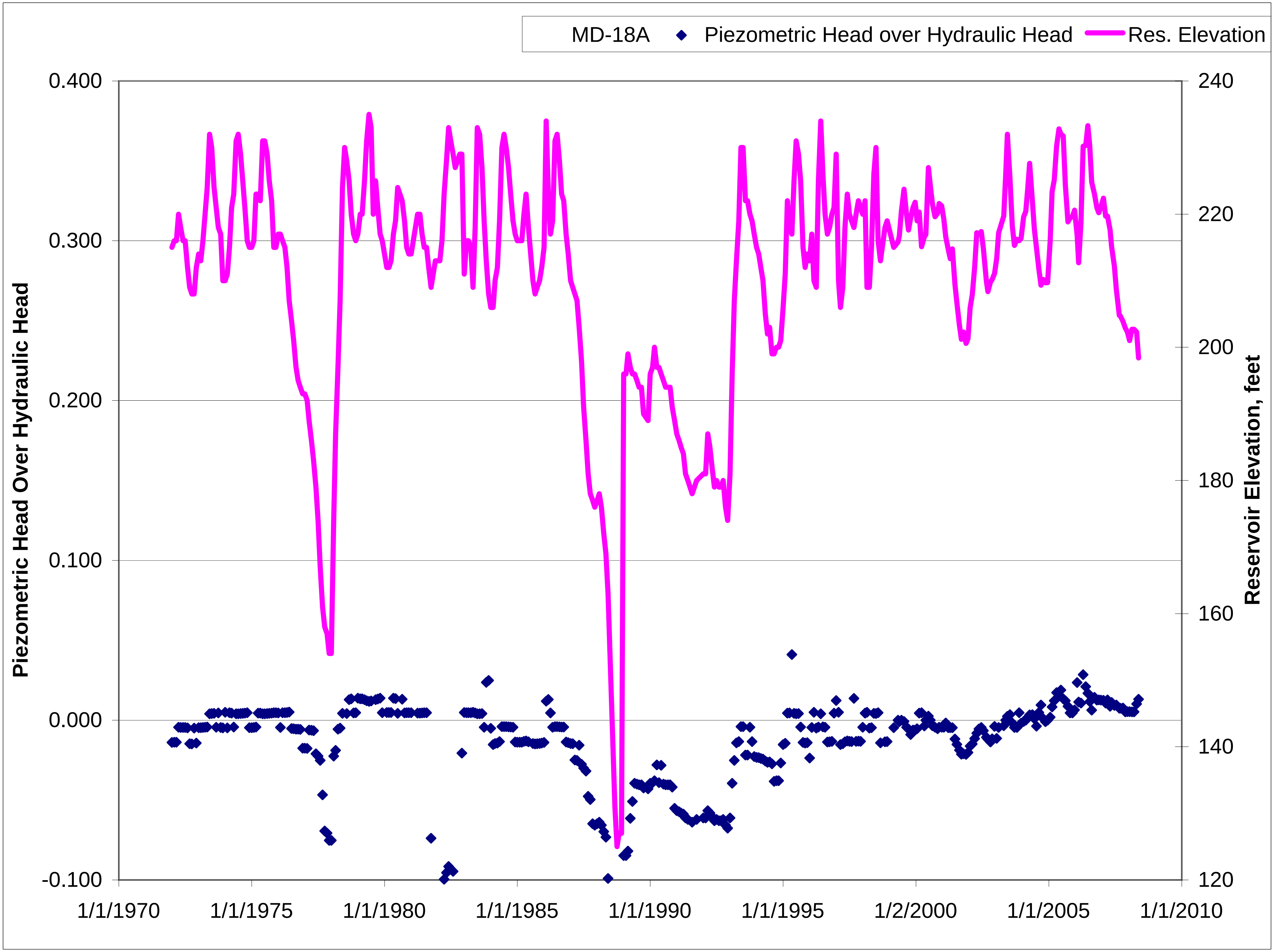

MD-16A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010


MD-16B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
-0.100
0.000
0.100
0.200
0.300
0.400
120
140
160
180
200
220
240
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010


MD-17A
Piezometric Head over Hydraulic Head
Res. Elevation
Reservoir Elevation, feet
240
220
200
180
160
140
120
Piezometric Head Over Hydraulic Head
0.400
0.300
0.200
0.100
0.000
-0.100
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010



MD-17B
Piezometric Head over Hydraulic Head
Res. Elevation
Reservoir Elevation, feet
Piezometric Head Over Hydraulic Head
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010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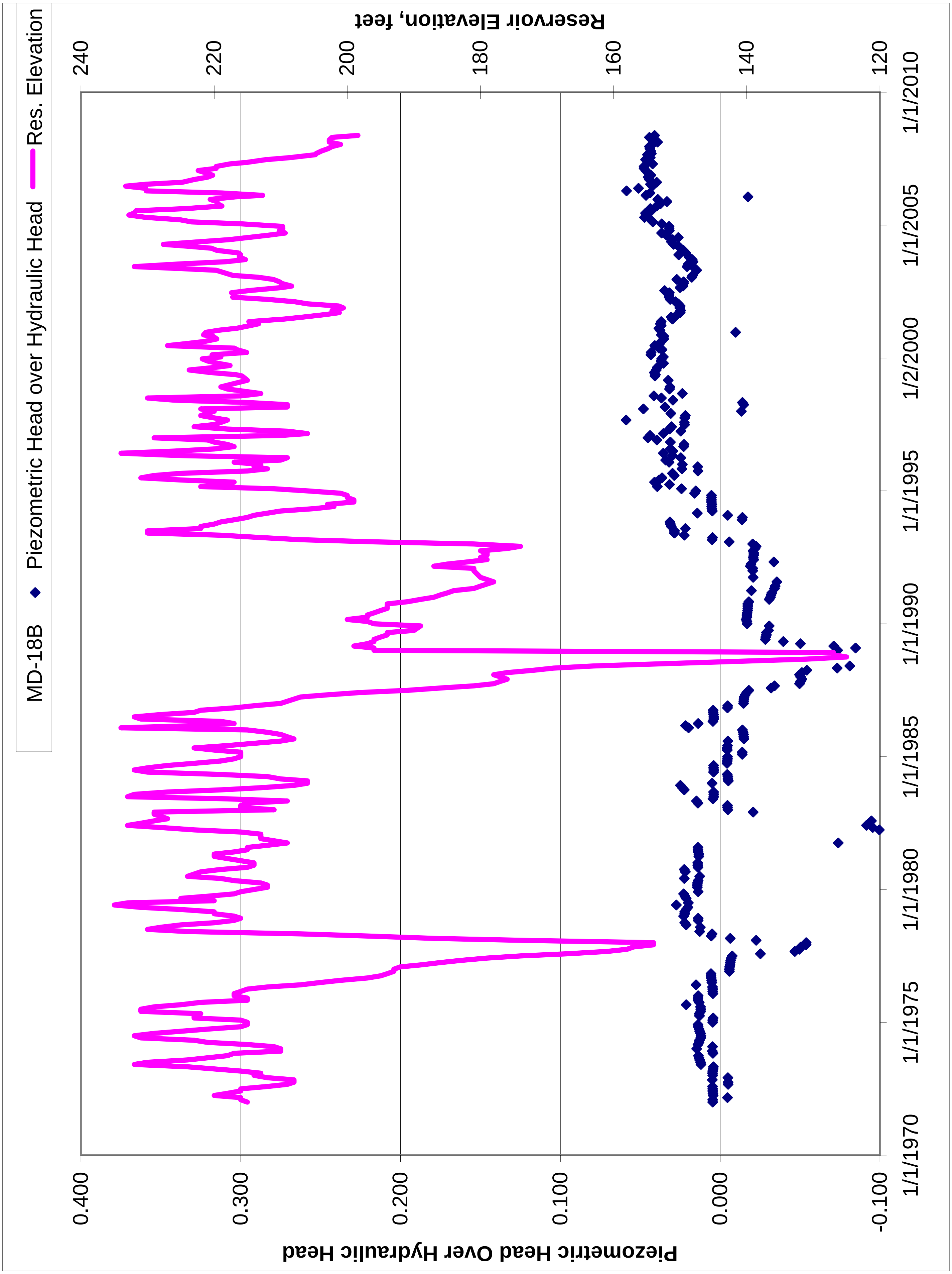
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010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Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
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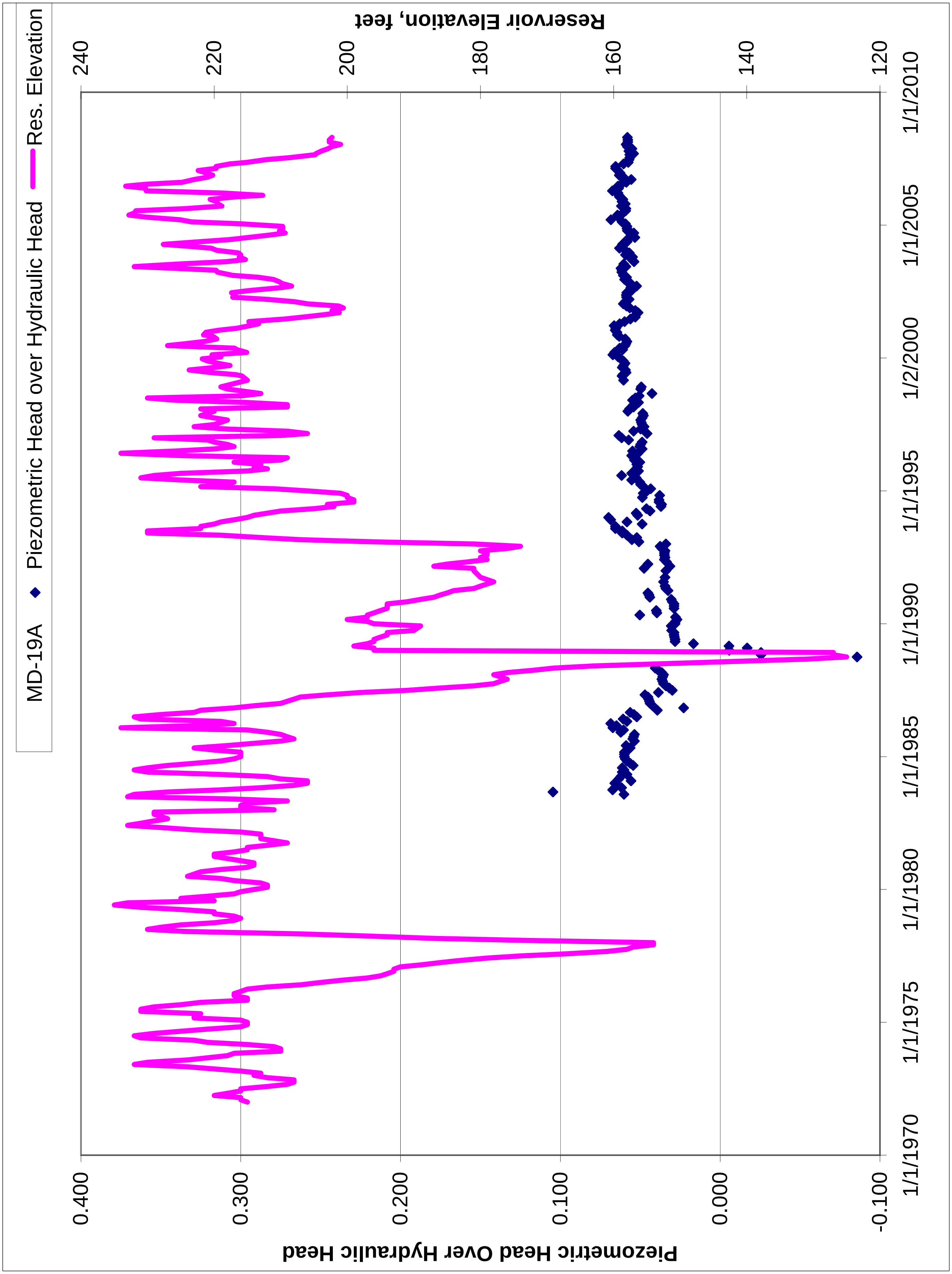
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010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Res. Elevation
Reservoir Elevation, feet
Piezometric Head Over Hydraulic Head
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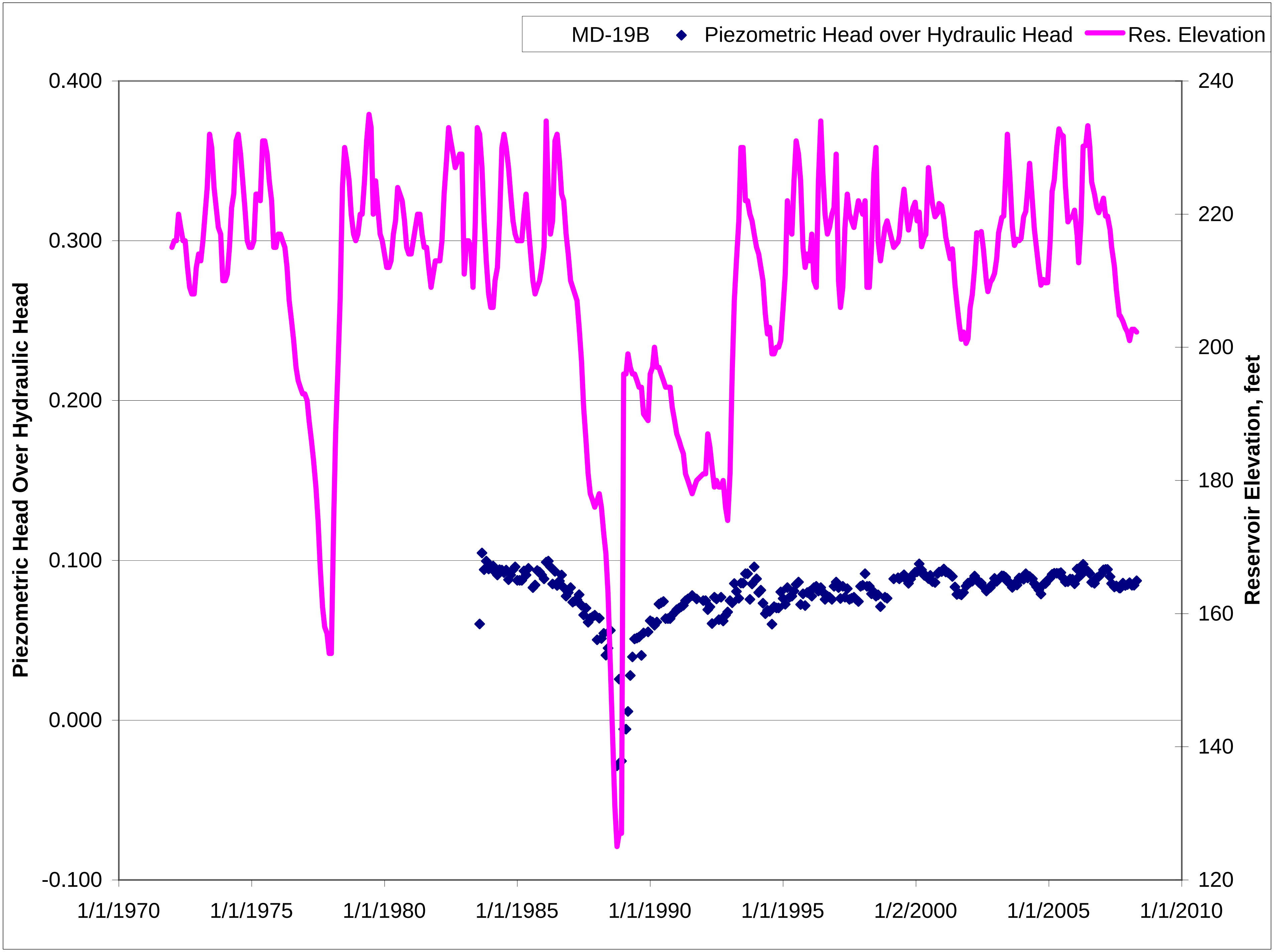
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010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Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
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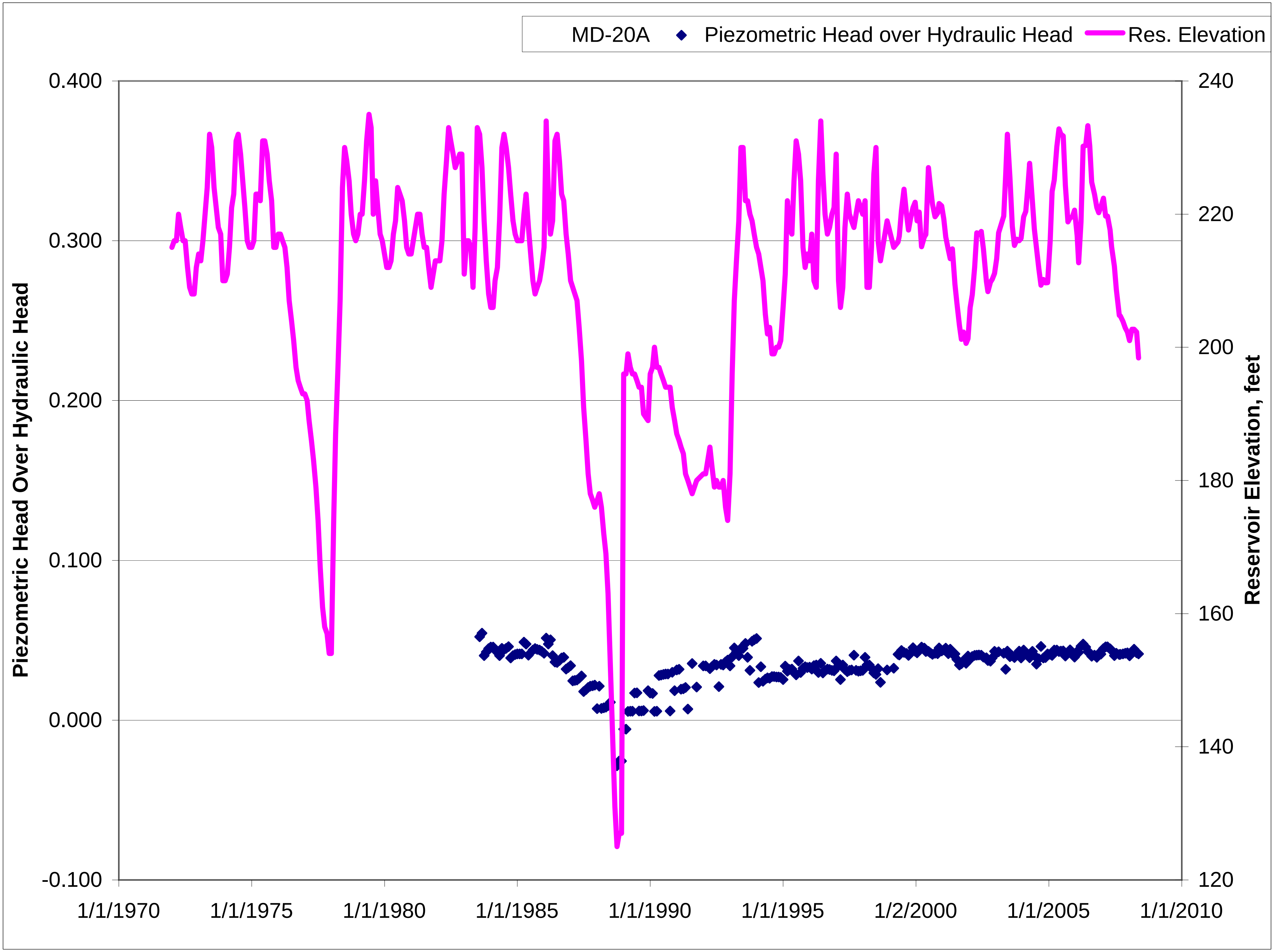
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010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Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
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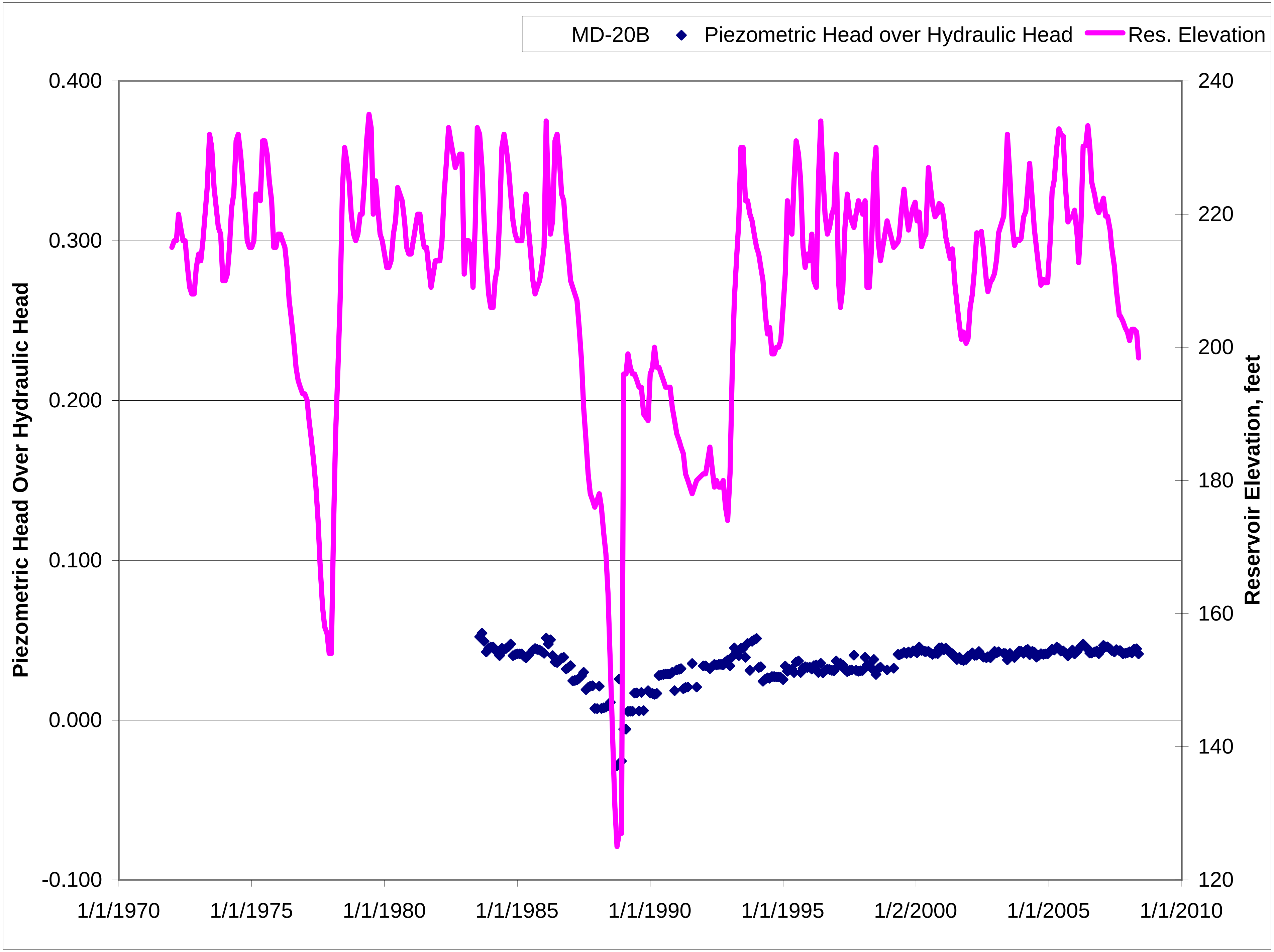
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010



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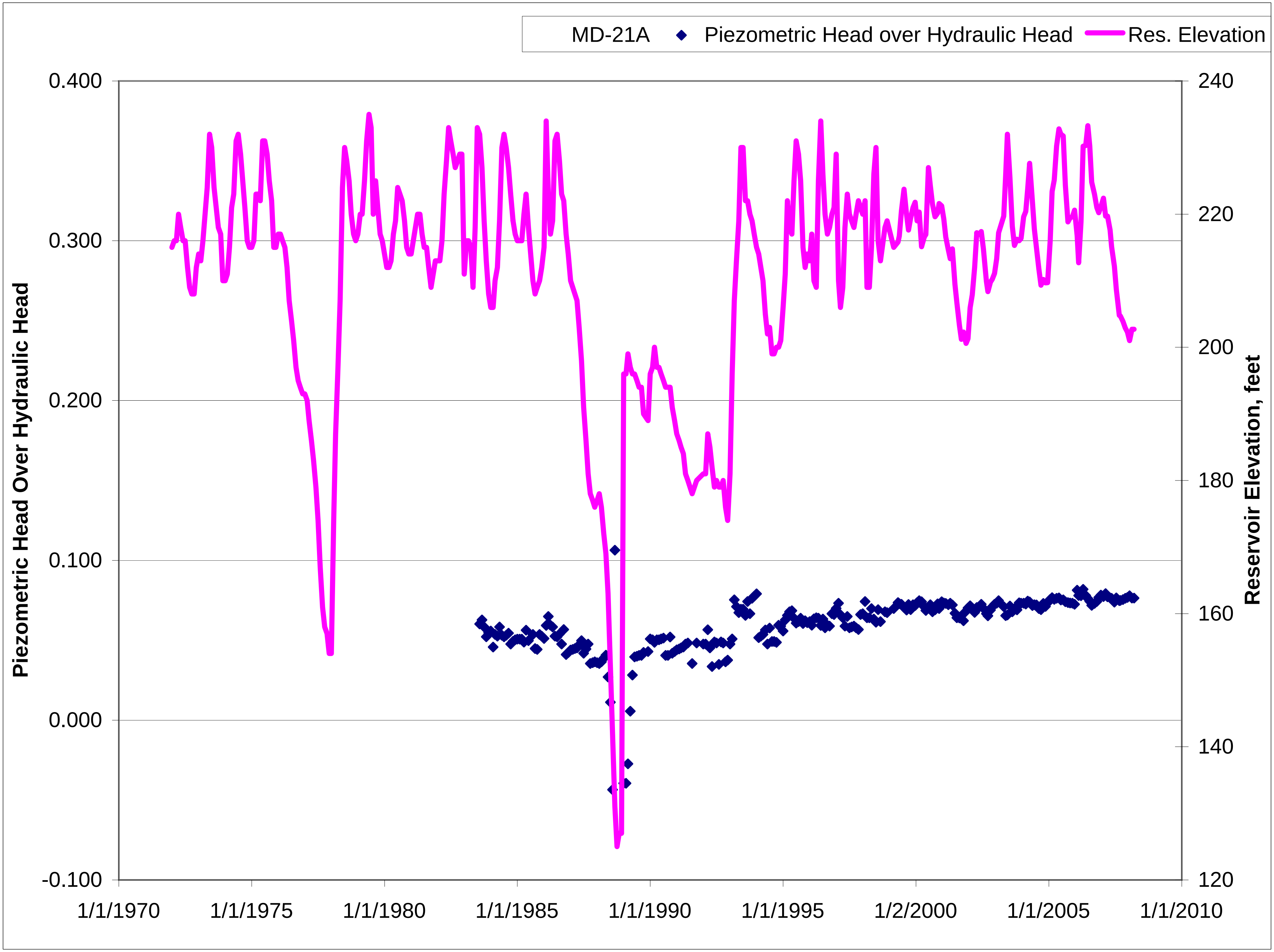
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010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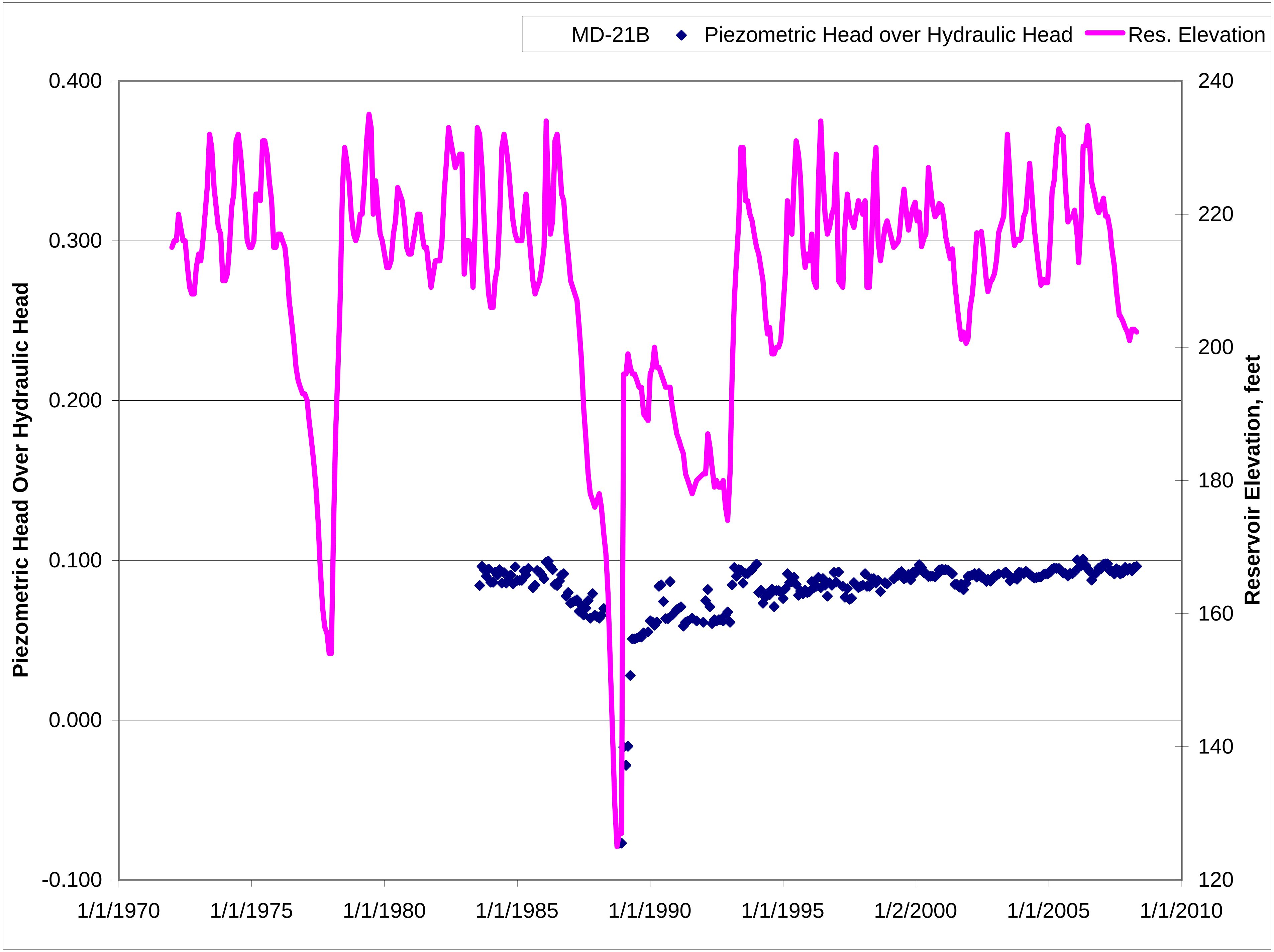
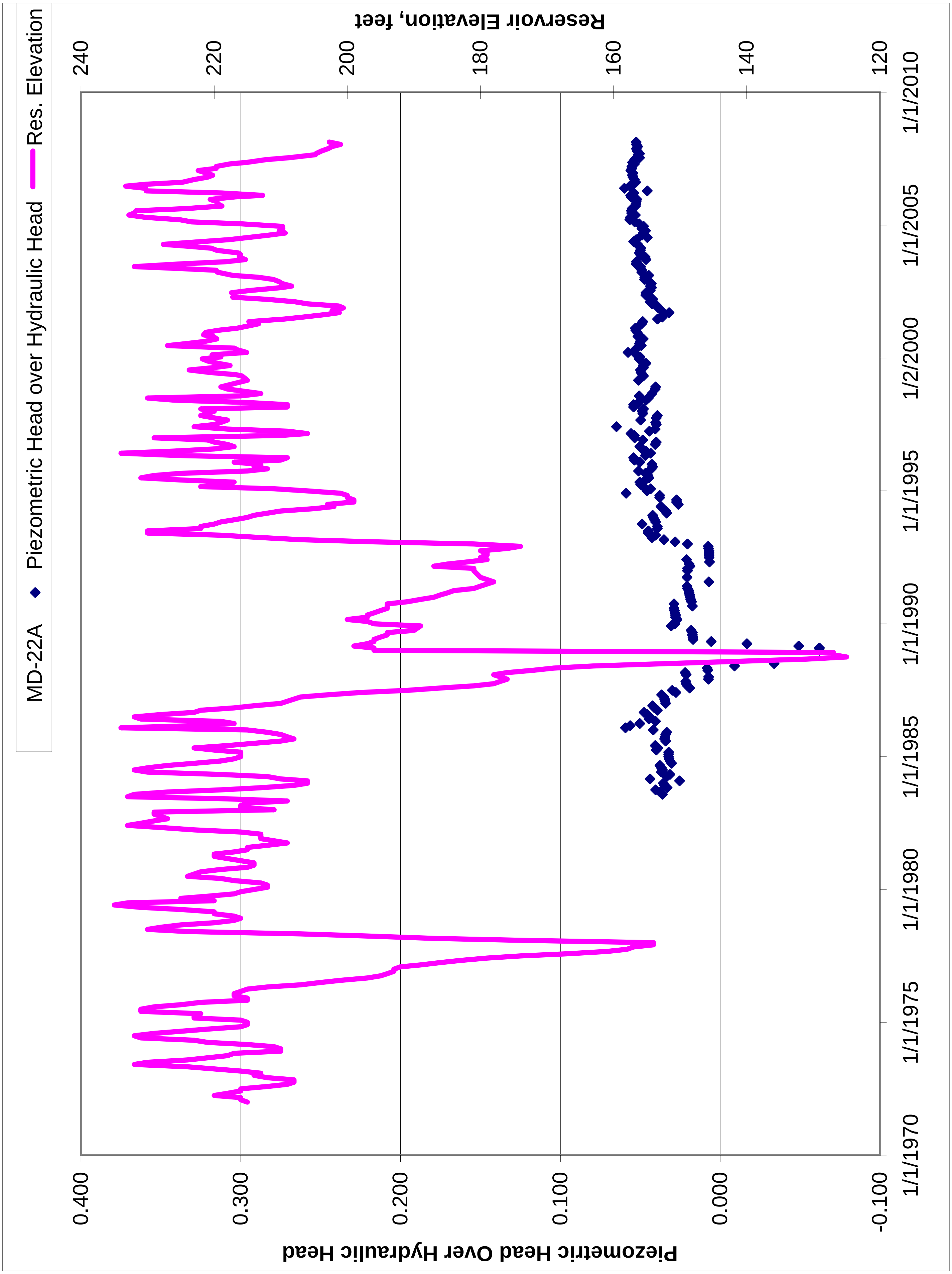
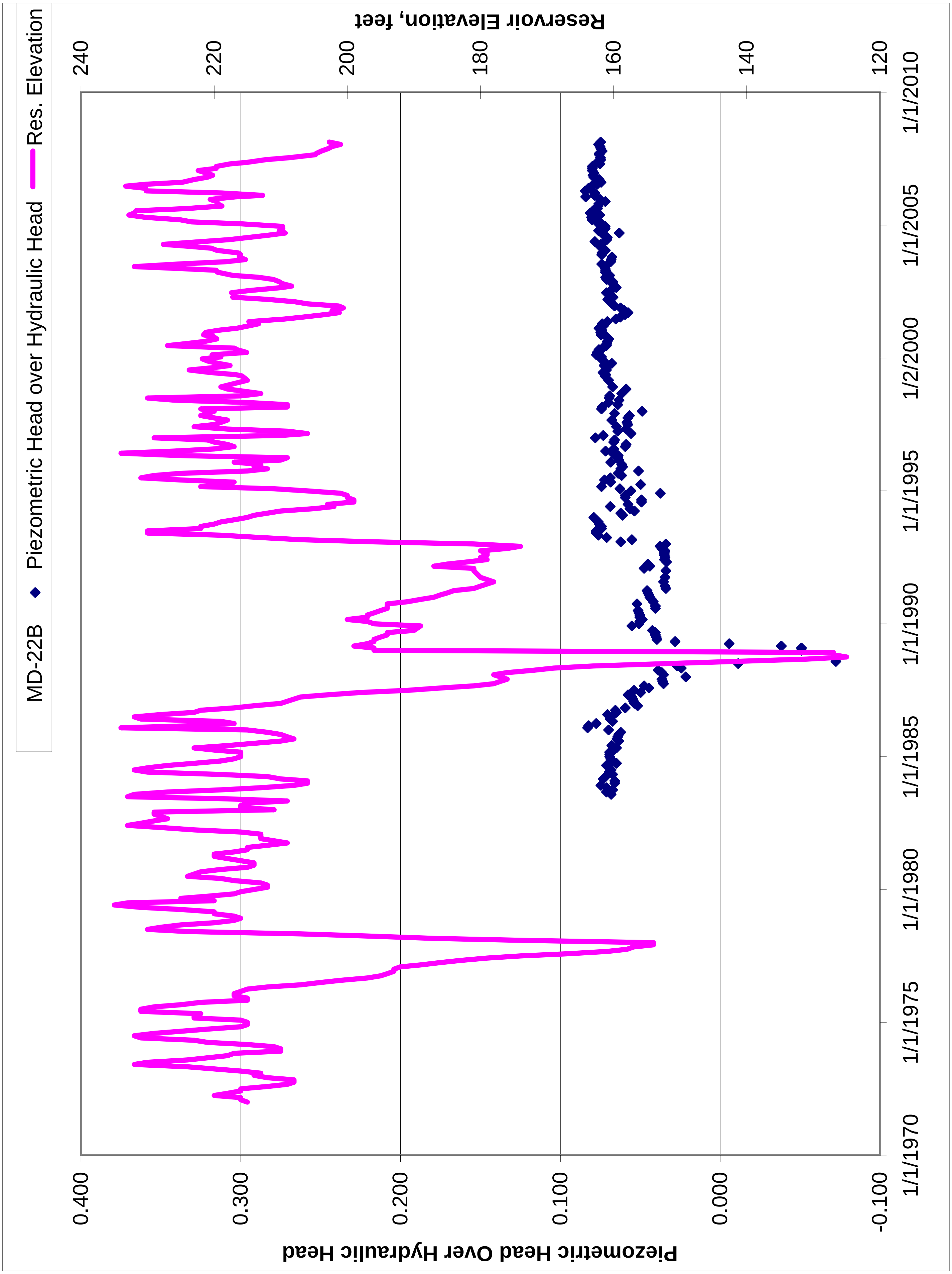
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010



MD-21B
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010


MD-22A
Piezometric Head over Hydraulic Head
Res. Elevation
Piezometric Head Over Hydraulic Head
Reservoir Elevation, feet
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010


MD-22B
Piezometric Head over Hydraulic Head
Res. Elevation
Reservoir Elevation, feet
Piezometric Head Over Hydraulic Head
0.400
0.300
0.200
0.100
0.000
-0.100
240
220
200
180
160
140
120
1/1/1970
1/1/1975
1/1/1980
1/1/1985
1/1/1990
1/1/1995
1/2/2000
1/1/2005
1/1/2010

APPENDIX E
DOWNHOLE SEISMIC VELOCITY MEASUREMENTS

INTRODUCTION

Downhole seismic velocity measurements were collected within eighteen wells located at the Main Dam, Dike 2, and Dike 5. The wells surveyed are shown on Figure E-1 and summarized on Table E-1 below.

TABLE E-1

SUMMARY OF WELLS SURVEYED

GENERAL LOCATION	WELLS SURVEYED
Main Dam Toe	MD-1A, MD-2A, MD-4A, MD-5A, MD-6A, MD-7A, MD-8B, MD-10A
Main Dam Embankment	DH-8
Main Dam Abutment	RA-3B, LA-3B
Dike 2	BH-5, OW-1, OW-4, OW-9, D2-7B
Dike 5	D5-3B, D5-4B

The purpose of the downhole seismic velocity measurements was to measure seismic pressure-wave (p-wave) and shear-wave (s-wave) velocities within the geologic materials adjacent to the boreholes. The geologic materials measured include the dredge tailings, alluvium, Mehrten formation, and Valley Springs formation.

SEISMIC DOWNHOLE MEASUREMENTS

Methods and Procedures

The seismic downhole method measures both pressure-wave (p-wave) and shear-wave (s-wave) velocities of the geologic-medium adjacent to a borehole by measuring the time required for these waves to travel from a surface source to a sensor fixed at a known depth within a borehole.

Downhole seismic measurements were performed on June 24 through June 26, 2008. A small diameter seismic probe was used to fit down the boreholes containing 1- to 2-inch diameter casings, and a larger diameter seismic probe was used within the boreholes containing 2- to 4-inch diameter casings. Each downhole seismic probe consisted of a triaxial geophone assembly which was pneumatically held against the boring walls. The seismic source consisted of a series of sledgehammer blows to a ground plate or a wood timber, which were offset 5-feet from the borehole. Two wave types (pressure and shear) were separately generated and measured. Vertical blows to the ground plate were used to produce p-waves, and horizontal blows to the wood timber were used to produce s-waves. Horizontal blows were made in opposite directions in order to record the characteristic opposite first motions of the s-waves to determine travel-times. The resulting seismic downhole data were recorded with a Geometrics S12 signal enhancing seismograph. Measurements were made at intervals of 2.5-feet within the areas mapped as dredge tailings/fine tailings and 5 to 10-feet elsewhere. Measurement accuracy was independent of the seismic probe used (i.e. small versus large diameter probe). Where data quality warranted, travel-times were picked to the nearest approximately 0.1 ms, especially within the areas mapped as dredge tailings/fine tailings. After correcting the travel time for the source offset, seismic-wave travel-times were plotted versus depth. Layer and interval velocities were calculated as the slope of lines drawn through the plotted data.

Results

Seismic measurements collected at the Main Dam, Dike 2, and Dike 5 are presented on Figures E-3 through E-13, Figures E-14 through E-18, and Figures E-19 and E-20, respectively. These figures show the measured p-wave and s-wave travel-times and depths; a plot of the p-wave and s-wave travel-times as a function of depth showing the interpreted layer velocities; interpreted p-wave and s-wave layer velocities and depth ranges; and calculated p-wave and s-wave interval velocities.

Tables E-2 through E-4 summarize the interpreted p-wave and s-wave layer velocities and depths that were collected at the Main Dam, Dike 2, and Dike 5, respectively. These tables summarize the velocity layers shown on Figures E-3 through and E-20. Also, a profile showing the shear wave velocities measured along the toe of the Main Dam is shown on Figure E-2.

A summary of the measured average s-wave velocity ranges for the various geologic units is presented on Table E-5. As shown on Table E-5, the Main Dam embankment has an s-wave velocity range of approximately 1,160 to 1,420 ft/sec with the uppermost 13-ft of DH-8 having an s-wave velocity of about 660 ft/sec. The tailings along the toe of the Main Dam have s-wave velocities that range between about 500 and 1,190 ft/sec, and a zone mapped as “Dredge Tailings?” within DH-8 between approximately 84 and 96 feet below the ground surface has an s-wave velocity of about 1,780 ft/sec. Also, a zone mapped as “Foundation Alluvium (Fine Dredge Tailings)” within DH-8 between 96 and 111 feet below the ground surface has an s-wave of approximately 980 ft/sec. The measured s-wave velocities of the older alluvium mapped within the area of Dike 2 range from about 690 to 1,330 ft/sec. Finally, the Mehrten and Valley Springs formations generally have similar s-wave velocities. The s-wave velocity of the Mehrten formation ranges from approximately 1,370 to 3,640 ft/sec, and the s-wave velocity of the Valley Springs formation ranges from approximately 2,370 to 3,810 ft/sec.

TABLE E-2

SUMMARY OF MAIN DAM SEISMIC VELOCITY LAYERS

BOREHOLE	GEOLOGIC UNIT	PRESSURE-WAVE Depth Range (ft)	PRESSURE-WAVE Velocity (ft/sec)	SHEAR-WAVE Depth Range (ft)	SHEAR-WAVE Velocity (ft/sec)
MD-1A	Dredge Tailings	0 to 10	1,450	0 to 6	1,040
		10 to 35	9,210	6 to 36	700
	Mehrten Fm	35 to 44	2,480	36 to 64	2,190
		44 to 64	5,060		
MD-2A	Dredge Tailings	0 to 8	1,960	0 to 21	580
		8 to 19	6,530		
	Mehrten Fm	19 to 27	3,730	21 to 62	2,260
		27 to 62	6,040		
MD-4A	Dredge Tailings	0 to 10	1,810	0 to 10	750
		10 to 25	6,530	10 to 28	600
	Mehrten Fm	25 to 38	4,650	28 to 38	2,080
MD-5A	Dredge Tailings	0 to 7	1,500	0 to 27	690
		7 to 27	6,730		
	Mehrten Fm	27 to 37	3.110	27 to 57	2,080
		37 to 57	4,780		
MD-6A	Dredge Tailings	0 to 5	2,800	2 to 10	760
		5 to 25	5,820	10 to 22	1,190
				22 to 29	700
	Mehrten Fm	25 to 30	1,290	29 to 32	2,370
MD-7A	Dredge Tailings	0 to 5	2,100	2 to 20	840
	Mehrten Fm	5 to 35	8,840	20 to 35	2,380
MD-8B	Mehrten Fm	0 to 9	3,590	4 to 69	2,490
		9 to 78	5,780		
MD-10	Dredge Tailings	0 to 9	2,550	0 to 7	820
				7 to 18	500
	Mehrten Fm	9 to 32	5,090	18 to 32	2,250
DH-8	Dam Embankment	0 to 78	2,060	0 to 13	660
		78 to 88	3,170	13 to 84	1,420
	Dredge Tailings?	88 to 113	4,930	84 to 96	1,780
	Alluvium or Fine Dredge Tailings			96 to 111	980
	Mehrten Fm			111 to 113	1,660
RA-3B	Mehrten Fm	5 to 70	4,840	8 to 70	3,200
LA-3B	Mehrten Fm	0 to 87	5,440	0 to 87	3,510

TABLE E-3

SUMMARY OF DIKE 2 SEISMIC VELOCITY LAYERS

BOREHOLE	GEOLOGIC UNIT	PRESSURE-WAVE		SHEAR-WAVE	
		Depth Range (ft)	Velocity (ft/sec)	Depth Range (ft)	Velocity (ft/sec)
BH-5	Dam Embankment	0 to 42	1,950	0 to 94	1,160
	Older Alluvium	42 to 64	2,890		
		64 to 94	4,880		
	Mehrten Fm	94 to 109	7,160	94 to 109	2,240
OW-1	Older Alluvium	0 to 5	1,190	0 to 25	930
		5 to 25	5,520		
OW-4	Older Alluvium	0 to 4	1,040	0 to 4	690
		4 to 44	7,310	4 to 34	1,150
				34 to 44	830
OW-9	Older Alluvium	0 to 9	2,250	0 to 24	1,330
	Mehrten Fm	9 to 69	4,860	24 to 74	2,720
D2-7B	Mehrten Fm	0 to 22	2,030	0 to 19	1,370
		22 to 69	8,030	19 to 69	3,640

TABLE E-4

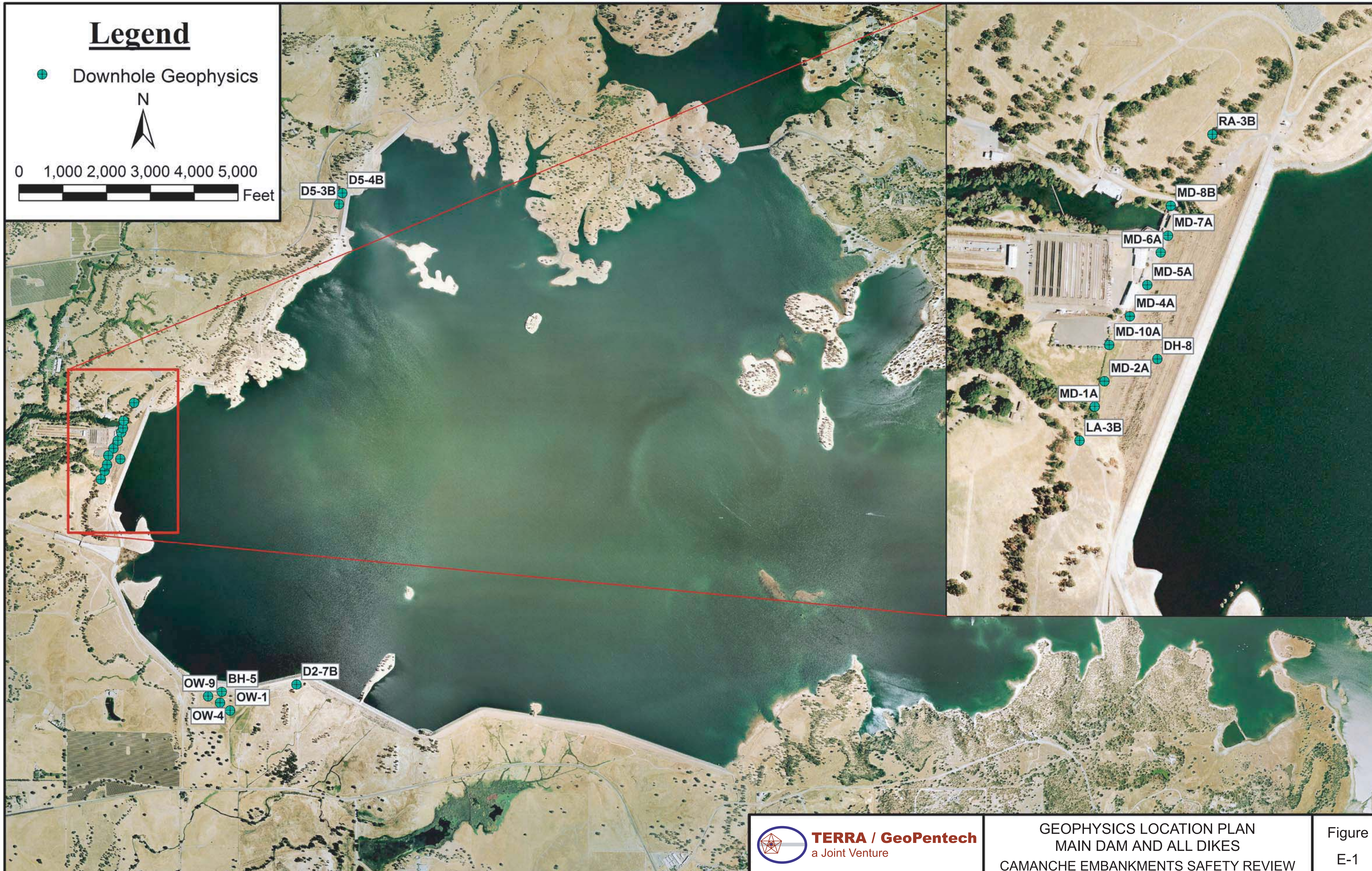
SUMMARY OF DIKE 5 SEISMIC VELOCITY LAYERS

BOREHOLE	GEOLOGIC UNIT	PRESSURE-WAVE		SHEAR-WAVE	
		Depth Range (ft)	Velocity (ft/sec)	Depth Range (ft)	Velocity (ft/sec)
D5-3B	Valley Springs Fm	0 to 24	3,120	0 to 24	2,370
		24 to 49	5,840	24 to 49	3,810
D5- 4B	Valley Springs Fm	0 to 25	5,060	0 to 25	2,550

TABLE E-5

SUMMARY OF SHEAR-WAVE VELOCITY LAYERS

GEOLOGIC UNIT	APPROXIMATE SHEAR-WAVE VELOCITY RANGE (ft/sec)
Dam Embankment (Main Dam and Dike 2)	1,160 to 1,420 (*660 upper 13-ft of DH-8)
Dredge Tailings (Toe Main Dam)	500 to 1,190
Dredge Tailings? (DH-8 - Main Dam)	1,780
Foundation Alluvium/Fine Tailings (DH-8 - Main Dam)	980
Older Alluvium (Dike 2)	690 to 1,330
Mehrten Fm (Main Dam and Dike 2)	1,370 to 3,640
Valley Springs Fm (Dike 5)	2,370 to 3,810



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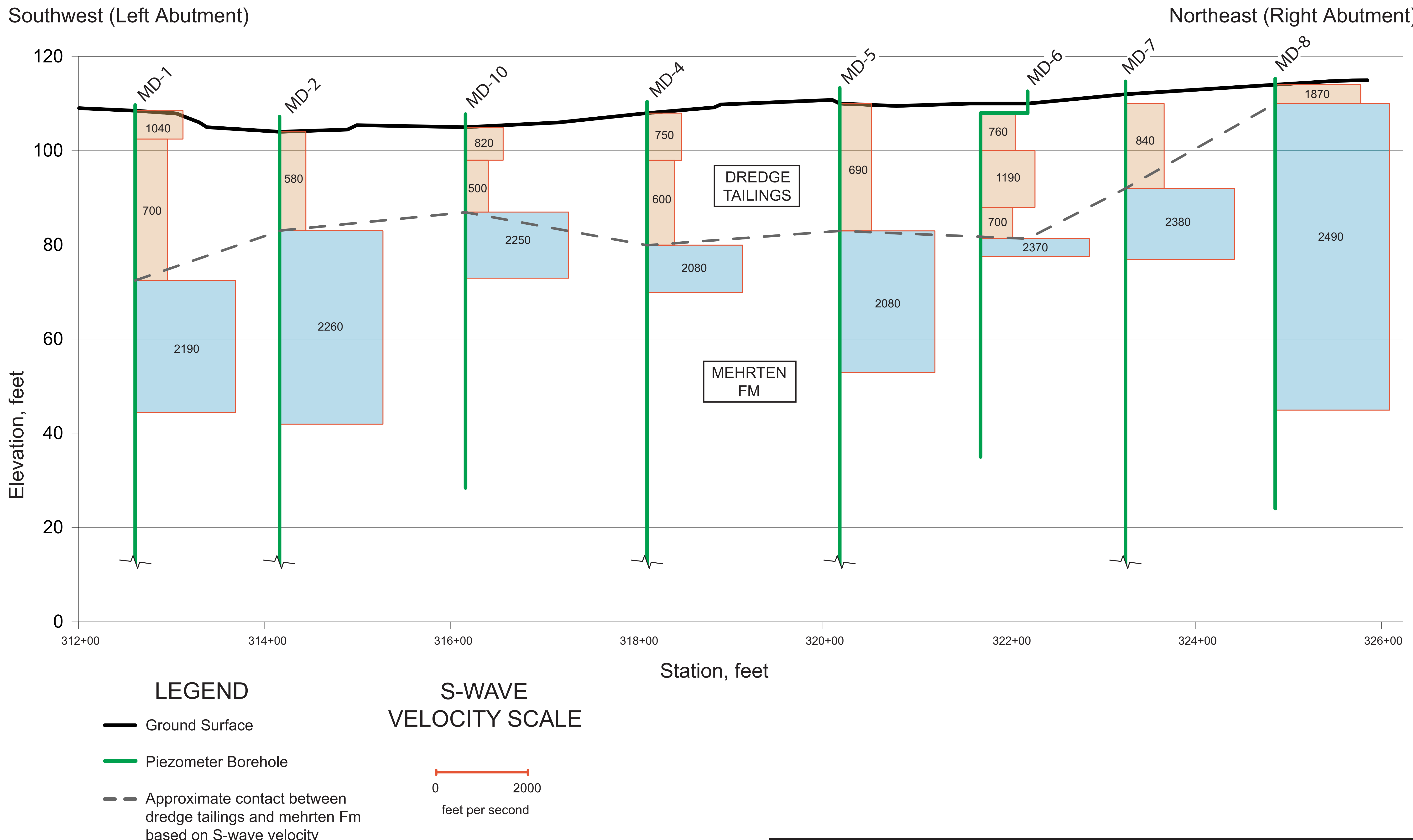
GEOPHYSICS LOCATION PLAN
MAIN DAM AND ALL DIKES
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-1

Rev. 1 02/05/10



TERRA / GeoPentech a Joint Venture	SHEAR WAVE VELOCITIES AT TOE OF MAIN DAM CAMANCHE EMBANKMENTS SAFETY REVIEW	Figure E-2

Rev. 3 02/05/10

Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	5
3.8	4.4	1	6.1	1	
6.3	6.2	1	7.7	12	
8.8	6.6	1	11.4	2	
11.3	6.9	2	14	2	
13.8	7.2	2	19.5	2	
16.3	7.4	2	22.8	2	
18.8	7.9	2	25.3	2	
21.3	8	2	27.7	2	
23.8	8.1	2	33.4	2	
26.3	8.2	2	36.8	2	
28.8	8.4	2	39	2	
31.3	8.6	2	41.5	2	
33.8	9	2	46.1	2	
36.3	9.2	3	49	23	
38.8	10.1	3	51	3	
43.8	12.2	34	53.5	3	
48.8	13	4	55.5	3	
53.8	14.4	4	57.5	3	
58.8	15.1	4	59	3	
63.8	16	4	62	3	

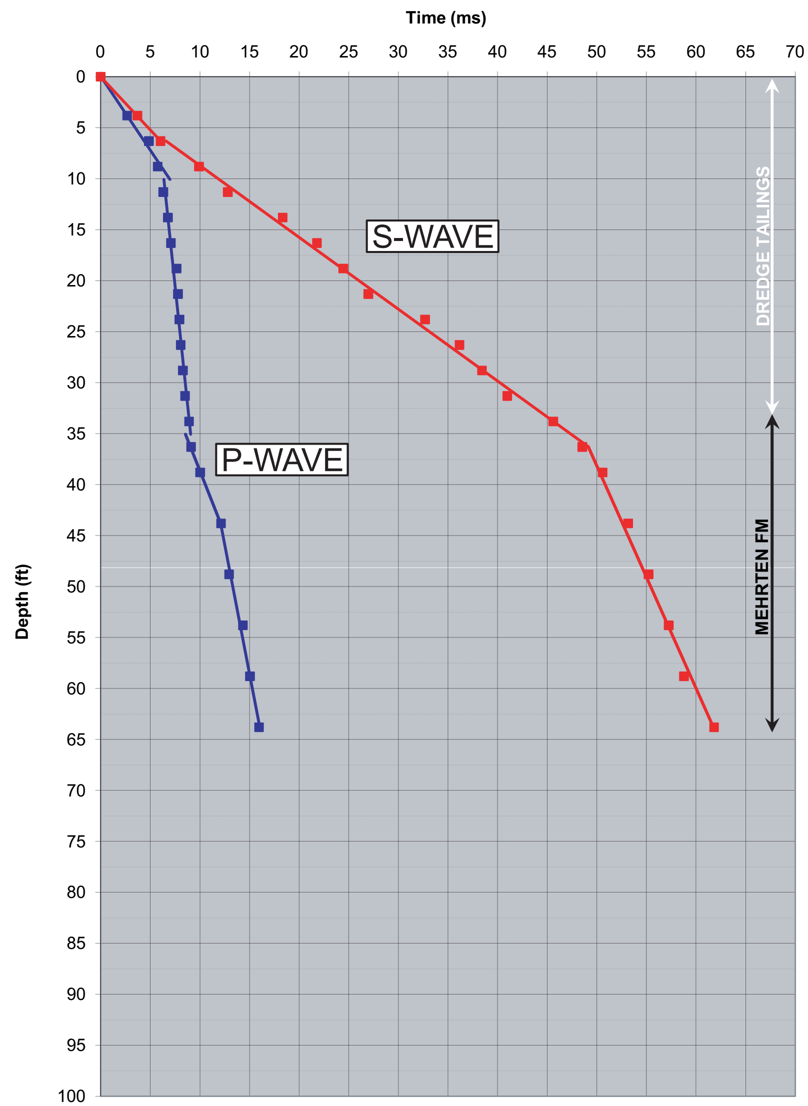
Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	1453	0 to 10	1043	0 to 6
2	9256	10 to 35	704	6 to 36
3	2473	35 to 44	2187	36 to 64
4	5062	44 to 64		
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 3.8	1427	1030
3.8 to 6.3	1139	1068
6.3 to 8.8	2834	644
8.8 to 11.3	4375	865
11.3 to 13.8	5441	452
13.8 to 16.3	8190	722
16.3 to 18.8	4465	943
18.8 to 21.3	16266	993
21.3 to 23.8	18029	437
23.8 to 26.3	19417	721
26.3 to 28.8	11338	1100
28.8 to 31.3	11567	978
31.3 to 33.8	6086	541
33.8 to 36.3	11858	851
36.3 to 38.8	2768	1225
38.8 to 43.8	2376	1943
43.8 to 48.8	6165	2432
48.8 to 53.8	3556	2448
53.8 to 58.8	7067	3258
58.8 to 63.8	5522	1654

Well Stickup = 1.2 ft
Ground Elevation = 108.5 ft

MD-1A DOWNHOLE SEISMIC MEASUREMENTS
CAMANCHE DAM



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DOWNHOLE SEISMIC MEASUREMENTS
MAIN DAM - MD-1A
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-3

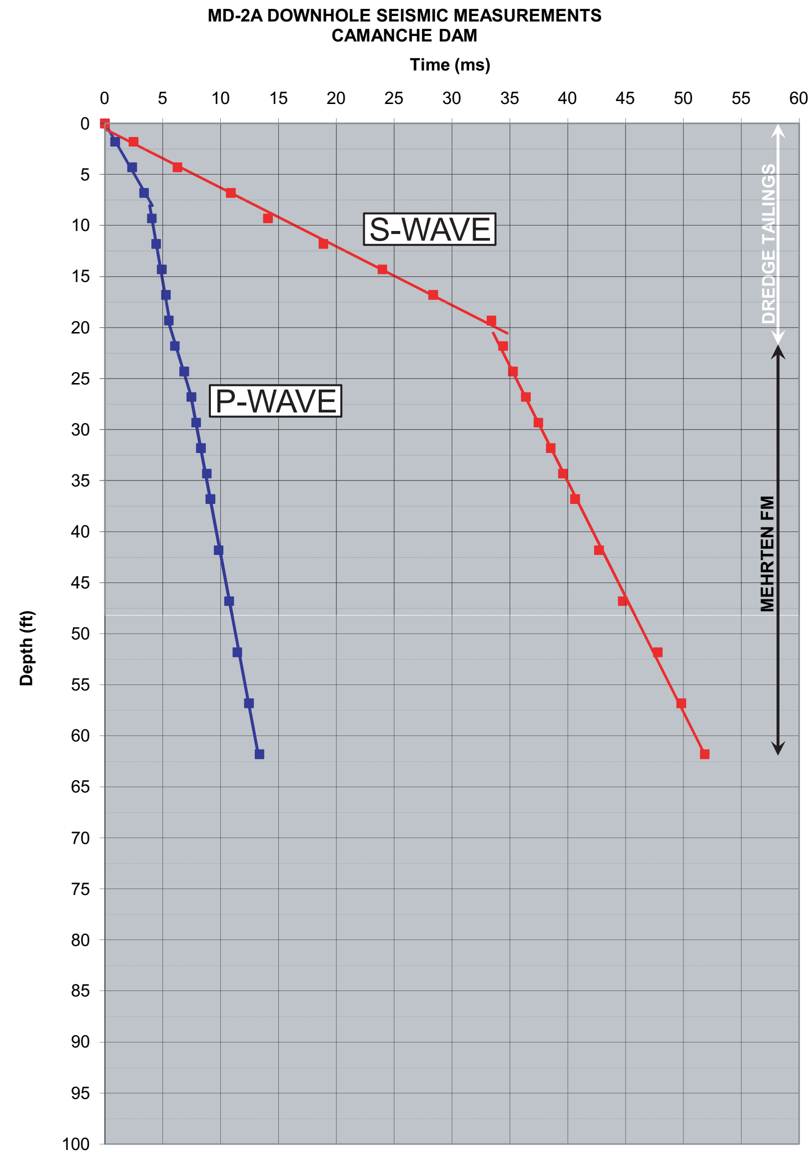
Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	5
1.8	2.6	1	7.3	1	
4.3	3.6	1	9.6	1	
6.8	4.2	1	13.5	1	
9.3	4.6	2	16	1	
11.8	4.8	2	20.5	1	
14.3	5.2	2	25.4	1	
16.8	5.5	2	29.6	1	
19.3	5.7	23	34.5	1	
21.8	6.2	3	35.3	2	
24.3	7	3	36	2	
26.8	7.6	34	37	2	
29.3	8	4	38	2	
31.8	8.4	4	39	2	
34.3	8.9	4	40	2	
36.8	9.2	4	41	2	
41.8	9.9	4	43	2	
46.8	10.8	4	45	2	
51.8	11.5	4	48	2	
56.8	12.5	4	50	2	
61.8	13.4	4	52	2	

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	1964	0 to 8	576	0 to 21
2	6532	8 to 19	2259	21 to 62
3	3723	19 to 27		
4	6044	27 to 62		
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 1.8	2044	728
1.8 to 4.3	1705	660
4.3 to 6.8	2412	542
6.8 to 9.3	3743	777
9.3 to 11.8	6793	523
11.8 to 14.3	5113	490
14.3 to 16.8	6889	569
16.8 to 19.3	10148	497
19.3 to 21.8	4760	2477
21.8 to 24.3	3074	2925
24.3 to 26.8	4067	2250
26.8 to 29.3	6025	2302
29.3 to 31.8	6067	2340
31.8 to 34.3	4913	2370
34.3 to 36.8	8082	2392
36.8 to 41.8	7006	2417
41.8 to 46.8	5501	2439
46.8 to 51.8	7063	1649
51.8 to 56.8	4975	2464
56.8 to 61.8	5528	2471

Well Stickup = 3.2 ft
Ground Elevation = 104 ft



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DOWNHOLE SEISMIC MEASUREMENTS
MAIN DAM - MD-2A
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure E-4

Rev. 3 02/05/10

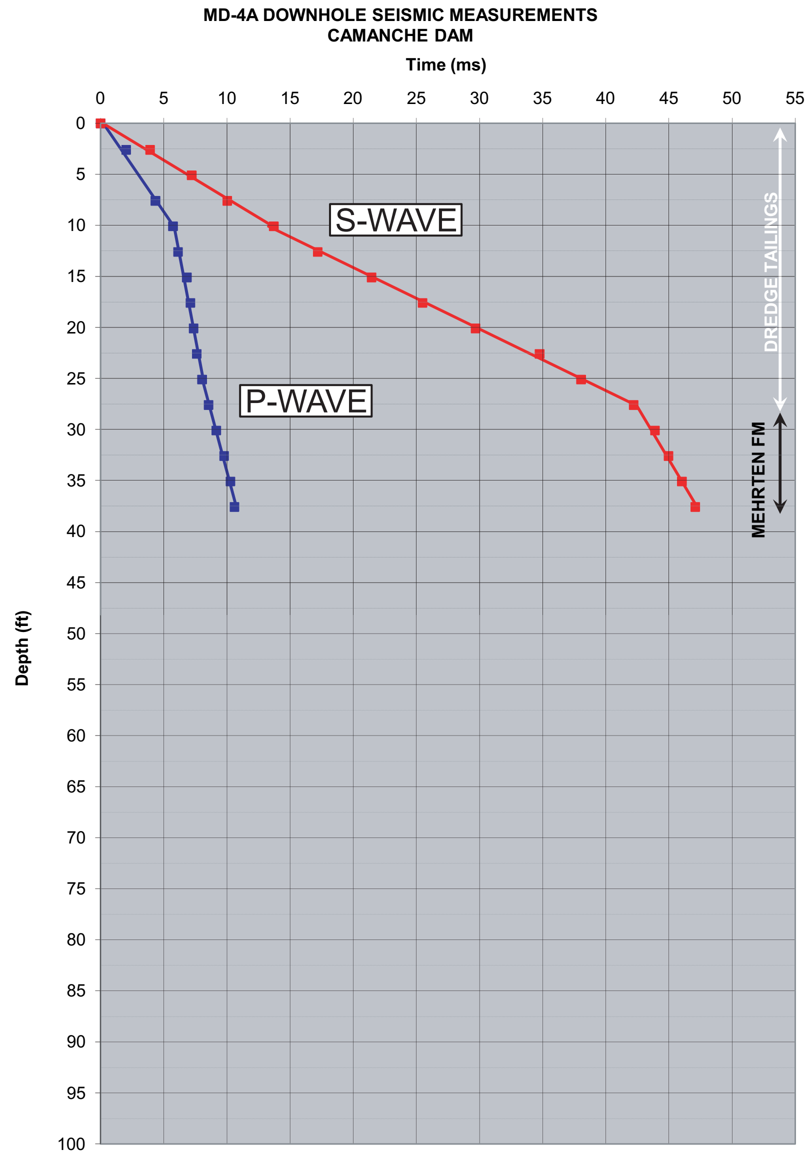
Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	5
2.6	4.4	1	8.5	1	
5.1			10.1	1	
7.6	5.2	1	12	1	
10.1	6.4	12	15.3	12	
12.6	6.6	2	18.5	2	
15.1	7.2	2	22.6	2	
17.6	7.4	2	26.5	2	
20.1	7.6	2	30.6	2	
22.6	7.8	2	35.6	2	
25.1	8.2	23	38.8	2	
27.6	8.7	3	42.9	23	
30.1	9.3	3	44.5	3	
32.6	9.9	3	45.5	3	
35.1	10.4	3	46.5	3	
37.6	10.7	3	47.5	3	

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	1808	0 to 10	750	0 to 10
2	6527	10 to 25	603	10 to 28
3	4654	25 to 38	2081	28 to 38
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 2.6	1281	663
2.6 to 5.1		760
5.1 to 7.6		889
7.6 to 10.1	1797	678
10.1 to 12.6	6266	718
12.6 to 15.1	3569	587
15.1 to 17.6	8825	619
17.6 to 20.1	9731	595
20.1 to 22.6	10390	494
22.6 to 25.1	5866	759
25.1 to 27.6	4820	601
27.6 to 30.1	4074	1483
30.1 to 32.6	4090	2324
32.6 to 35.1	4897	2356
35.1 to 37.6	8050	2380

Well Stickup = 2.4 ft
Ground Elevation = 108 ft



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DOWNHOLE SEISMIC MEASUREMENTS
MAIN DAM - MD-4A
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure E-5

Rev. 3 02/05/10

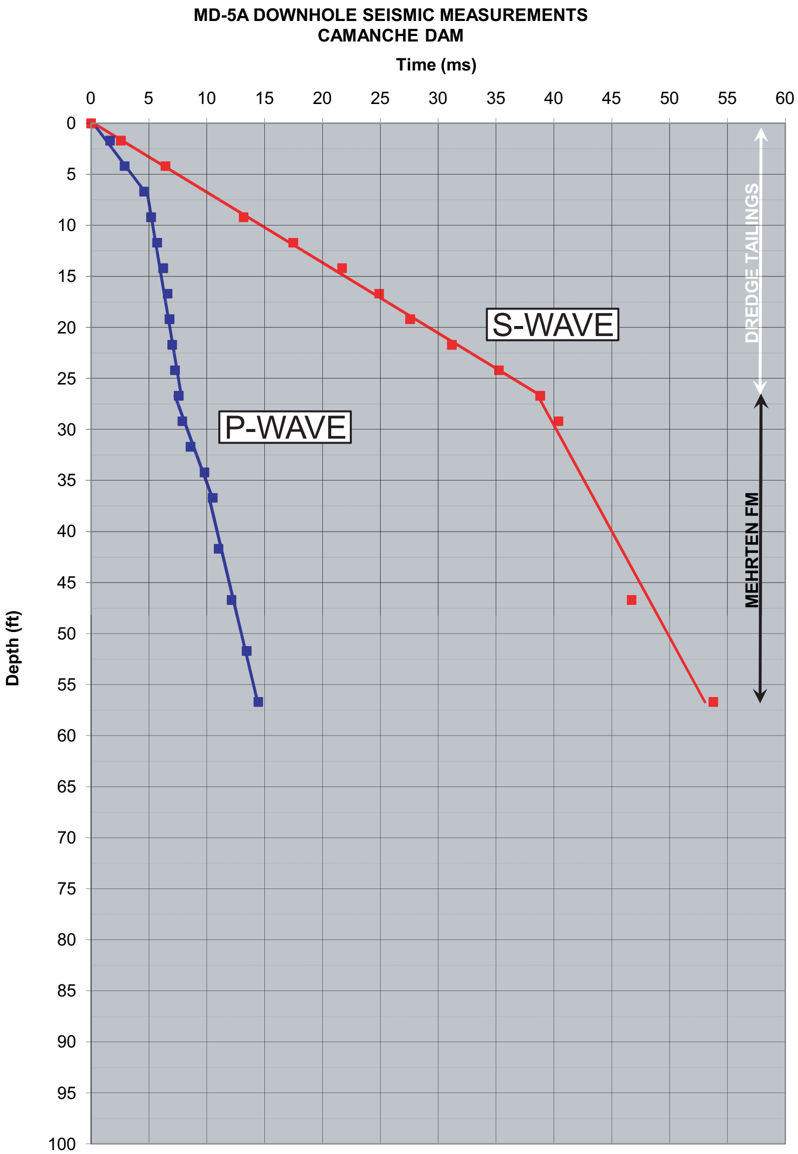
Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	5
1.7	5.1	1	8	1	
4.2	4.5	1	10	1	
6.7	5.7	12			
9.2	5.9	2	15	1	
11.7	6.2	2	19	1	
14.2	6.6	2	23	1	
16.7	6.9	2	26	1	
19.2	7	2	28.5	1	
21.7	7.2	2	32	1	
24.2	7.4	2	36	1	
26.7	7.7	23	39.5	12	
29.2	8	3	41	2	
31.7	8.7	3			
34.2	9.9	3			
36.7	10.6	34			
41.7	11.1	4			
46.7	12.2	4	47	2	
51.7	13.5	4			
56.7	14.5	4	54	2	

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	1504	0 to 7	690	0 to 27
2	6725	7 to 27	2080	27 to 57
3	3105	27 to 37		
4	4775	37 to 57		
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 1.7	1036	660
1.7 to 4.2	1996	648
4.2 to 6.7	1494	
6.7 to 9.2	4060	
9.2 to 11.7	4832	582
11.7 to 14.2	4770	592
14.2 to 16.7	6498	778
16.7 to 19.2	15246	935
19.2 to 21.7	10327	694
21.7 to 24.2	10833	614
24.2 to 26.7	7776	700
26.7 to 29.2	7891	1576
29.2 to 31.7	3528	
31.7 to 34.2	2080	
34.2 to 36.7	3536	
36.7 to 41.7	9651	
41.7 to 46.7	4506	
46.7 to 51.7	3827	
51.7 to 56.7	4967	

Well Stickup = 3.3 ft
Ground Elevation = 110 ft



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DOWNHOLE SEISMIC MEASUREMENTS
MAIN DAM - MD-5A
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure E-6

Rev. 3 02/05/10

Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer
0	0	1	0	1
2.4	2	1	4.5	12
4.9	2.5	12	8	2
7.4	2.5	2	10.3	2
9.9	2.4	2	13.4	23
12.4	2.7	2	15.2	3
14.9	2.8	2	16.8	3
17.4	3.7	2	19	3
19.9	4.1	2	21	3
22.4	4.3	2	23	34
24.9	5.2	23	26	4
27.4	7	3	30	4
29.9	9.1	34	32.5	5
32.4	9.9	4	33.5	5

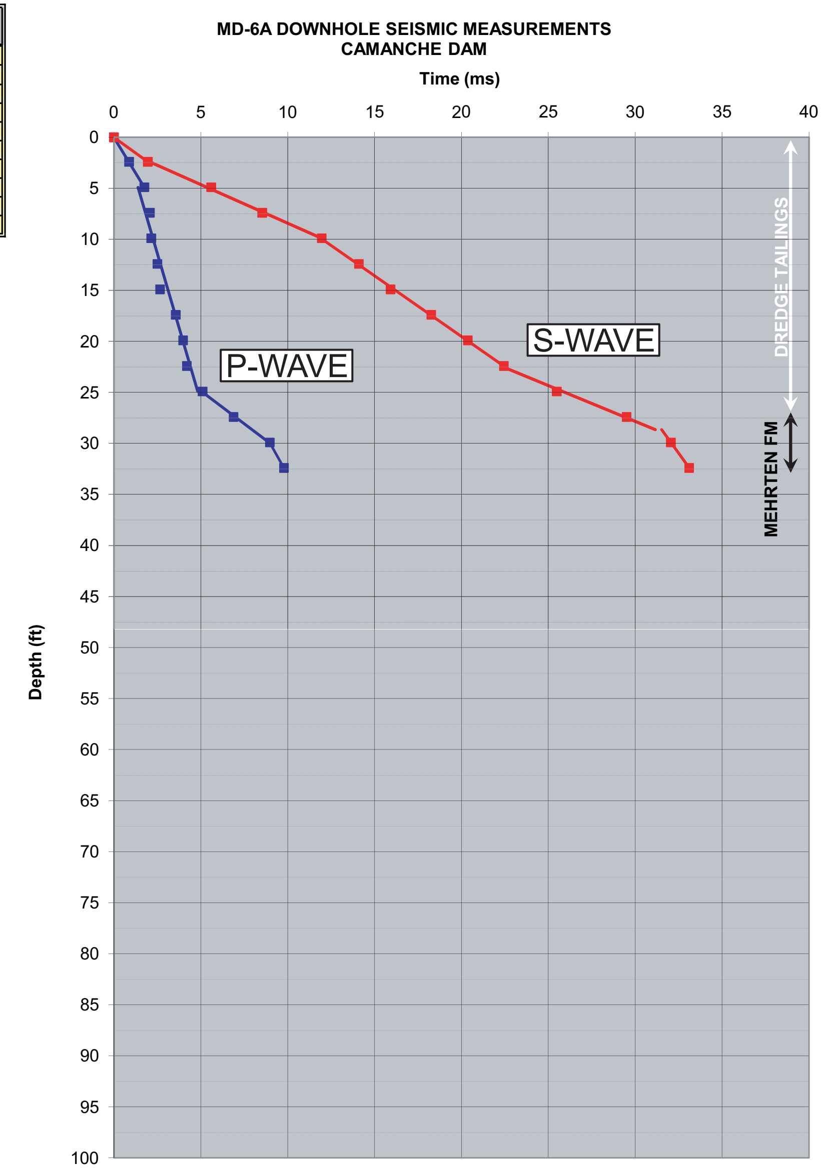
Offset (ft)
5

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	2800	0 to 5	1232	0 to 2
2	5818	5 to 25	757	2 to 10
3	1287	25 to 30	1188	10 to 22
4	3091	30 to 32	703	22 to 28.65
5			2374	29 to 32.4
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 2.4	2773	1232
2.4 to 4.9	2827	685
4.9 to 7.4	7772	852
7.4 to 9.9	35307	730
9.9 to 12.4	6910	1170
12.4 to 14.9	16619	1366
14.9 to 17.4	2773	1071
17.4 to 19.9	5948	1187
19.9 to 22.4	11347	1202
22.4 to 24.9	2773	821
24.9 to 27.4	1398	622
27.4 to 29.9	1197	983
29.9 to 32.4	3091	2374

Well Stickup = 2.6 ft
Ground Elevation = 110 ft



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DOWNHOLE SEISMIC MEASUREMENTS
MAIN DAM - MD-6A
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-7

Rev. 3 02/05/10

Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer
0	0	1	0	1
2.3	2.7	1	4	12
4.8	3.3	12	6.5	2
7.3	3.5	2	8.5	2
9.8	3.8	2	11	2
12.3	3.8	2	14	2
14.8	4.1	2	17.5	2
17.3	4.4	2	20	2
19.8	4.5	2		
22.3	4.7	2	23.5	3
24.8			24.5	3
27.3			25.5	3
29.8			26	3
32.3			27.5	3
34.8	5.8	2	28.5	3

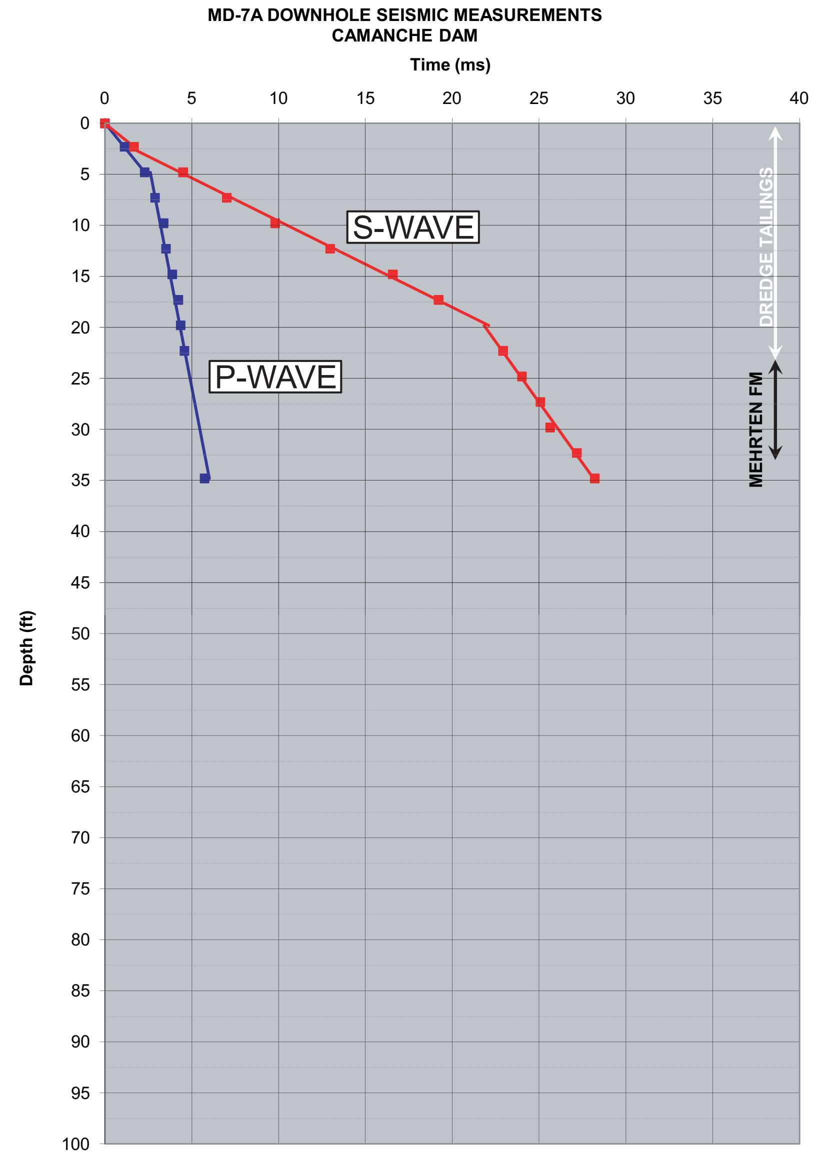
Offset (ft)
5

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	2101	0 to 5	1376	0 to 2
2	8835	5 to 35	844	2 to 20
3			2378	20 to 35
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 2.3	2038	1376
2.3 to 4.8	2161	883
4.8 to 7.3	4151	996
7.3 to 9.8	5027	897
9.8 to 12.3	18468	788
12.3 to 14.8	6867	693
14.8 to 17.3	7295	949
17.3 to 19.8	18377	
19.8 to 22.3	11206	
22.3 to 24.8		2302
24.8 to 27.3		2345
27.3 to 29.8		4474
29.8 to 32.3		1629
32.3 to 34.8		2418

Well Stickup = 2.7 ft
Ground Elevation = 112 ft



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DOWNHOLE SEISMIC MEASUREMENTS
MAIN DAM - MD-7A
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-8

Rev. 3 02/05/10

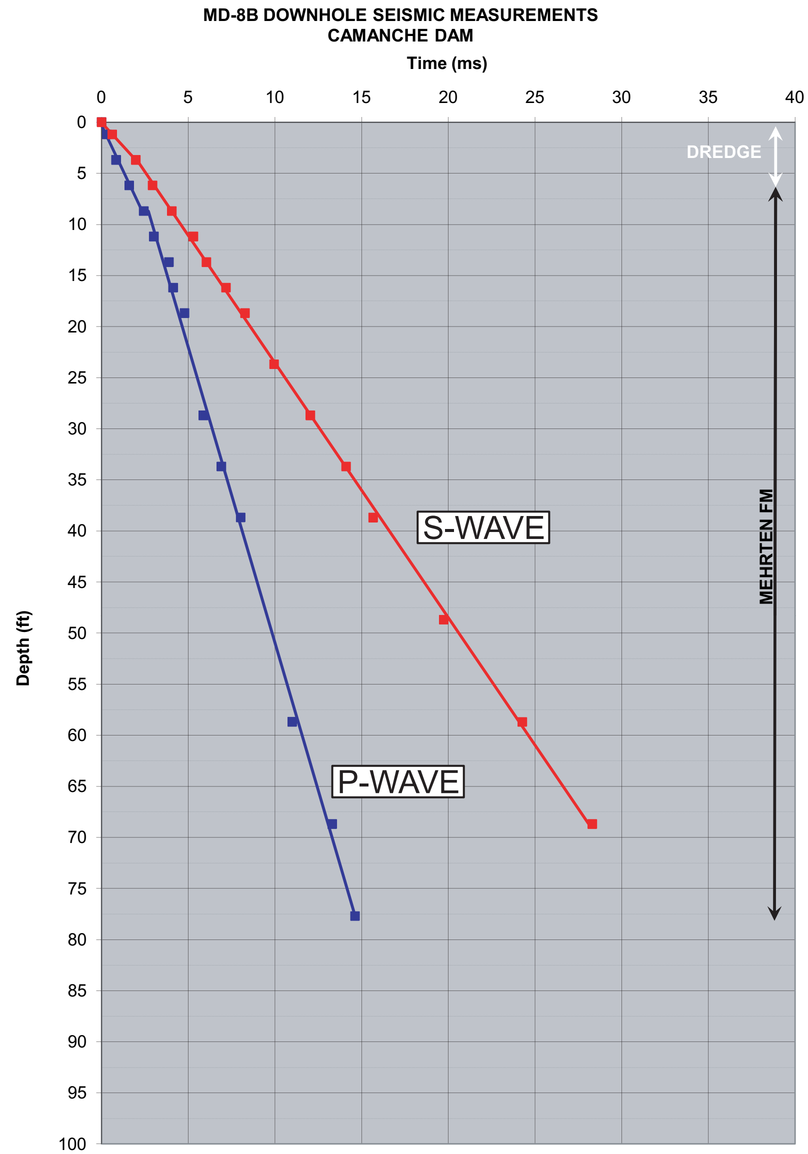
Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	8
1.2	2	1	4.1	1	
3.7	2	1	4.7	12	
6.2	2.6	1	4.8	2	
8.7	3.3	12	5.5	2	
11.2	3.7	2	6.5	2	
13.7	4.5	2	7	2	
16.2	4.6	2	8	2	
18.7	5.2	2	9	2	
23.7			10.5	2	
28.7	6.1	2	12.5	2	
33.7	7.1	2	14.5	2	
38.7	8.2	2	16	2	
48.7			20	2	
58.7	11.1	2	24.5	2	
68.7	13.4	2	28.5	2	
77.7	14.7	2			

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	3593	0 to 9	1868	0 to 4
2	5781	9 to 78	2493	4 to 69
3				
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 1.2	4045	1973
1.2 to 3.7	4605	1832
3.7 to 6.2	3319	2584
6.2 to 8.7	2989	2256
8.7 to 11.2	4298	2015
11.2 to 13.7	2857	3309
13.7 to 16.2	10481	2216
16.2 to 18.7	3809	2270
18.7 to 23.7		2987
23.7 to 28.7		2390
28.7 to 33.7	4845	2419
33.7 to 38.7	4456	3204
38.7 to 48.7		2459
48.7 to 58.7		2203
58.7 to 68.7	4326	2479
68.7 to 77.7	6856	

Well Stickup = 1.3 ft
Ground Elevation = 114 ft



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DOWNHOLE SEISMIC MEASUREMENTS
MAIN DAM - MD-8B
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure E-9

Rev. 3 02/05/10

Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer
0	0	1	0	1
4.2	2	1	7.8	1
6.7	2.8	1	10.2	12
9.2	4.1	12	15	2
11.7	4.9	2	18.4	2
14.2	5.2	2	24	2
16.7	5.3	2	29.7	2
19.2	5.7	2	31	3
21.7	5.9	2	32	3
24.2	6.4	2	33	3
26.7	7.2	2	34	3
29.2	7.9	2	35	3
31.7	8.3	2	36	3

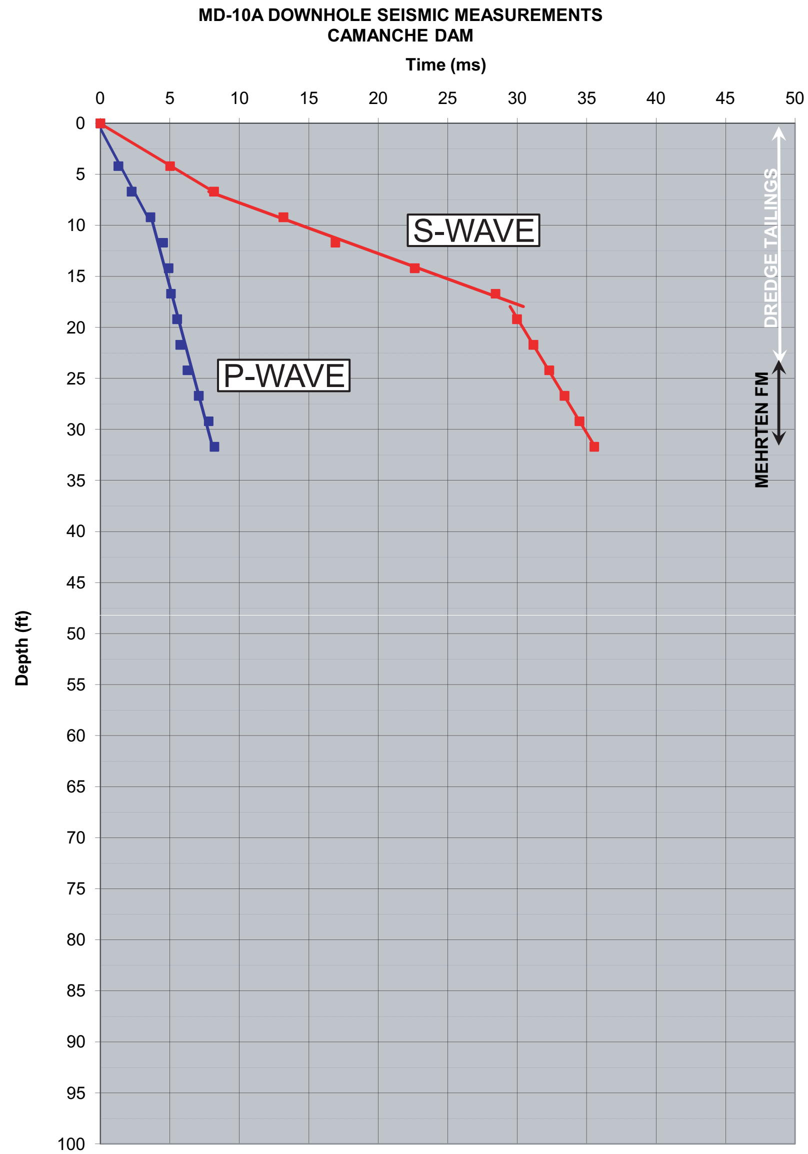
Offset (ft)
5

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	2553	0 to 9	821	0 to 7
2	5086	9 to 32	497	7 to 18
3			2251	18 to 32
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 4.2	3265	837
4.2 to 6.7	2611	792
6.7 to 9.2	1840	500
9.2 to 11.7	2767	668
11.7 to 14.2	6265	437
14.2 to 16.7	14493	430
16.7 to 19.2	5698	1616
19.2 to 21.7	10715	2112
21.7 to 24.2	4824	2204
24.2 to 26.7	3089	2269
26.7 to 29.2	3523	2317
29.2 to 31.7	6068	2353

Well Stickup = 0.8 ft
Ground Elevation = 105 ft



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DOWNHOLE SEISMIC MEASUREMENTS
MAIN DAM - MD-10A
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-10

Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	4.5
8	5	1	14	1	
18	6.3	1	24.8	2	
28	14.4	1	32.7	2	
38	19.2	1	37	2	
48	23.8	1	45.3	2	
58	28.8	1	52.3	2	
68	32.5	1	59.7	2	
78	36.5	12	66.4	2	
80.5	37.3	2	68.6	2	
83	37.9	2	69.5	2	
85.5	38.7	2	71.2	3	
88	39.7	23	72.3	3	
90.5	40.2	3	73.7	3	
93	41	3	75.3	3	
95.5	41.5	3	76.7	34	
98	42.1	3	79.5	4	
100.5	42.6	3	82.6	4	
103	43.1	3	84.5	4	
105.5	43.3	3	87.2	4	
108	43.7	3	89.4	4	
110.5	44.3	3	92.4	45	
113	44.9	3	93.9	5	

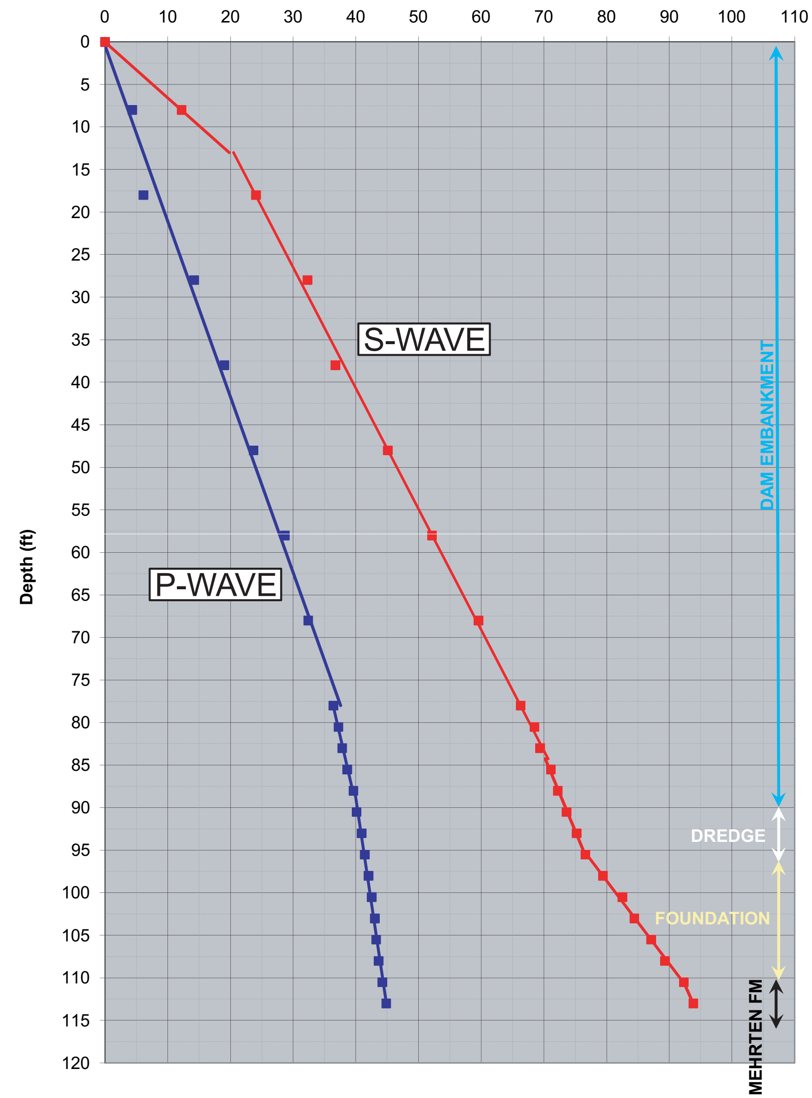
Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	2064	0 to 78	656	0 to 13
2	3173	78 to 88	1420	13 to 84
3	4925	88 to 113	1775	84 to 96
4			976	96 to 110.5
5			1664	111 to 113
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 8.0	1836	656
8.0 to 18.0	5701	843
18.0 to 28.0	1234	1216
28.0 to 38.0	2062	2243
38.0 to 48.0	2160	1196
48.0 to 58.0	1993	1420
58.0 to 68.0	2692	1347
68.0 to 78.0	2494	1488
78.0 to 80.5	3115	1135
80.5 to 83.0	4149	2762
83.0 to 85.5	3117	1468
85.5 to 88.0	2496	2264
88.0 to 90.5	4978	1781
90.5 to 93.0	3118	1560
93.0 to 95.5	4981	1782
95.5 to 98.0	4155	892
98.0 to 100.5	4983	806
100.5 to 103.0	4984	1314
103.0 to 105.5	12392	925
105.5 to 108.0	6227	1135
108.0 to 110.5	4158	833
110.5 to 113.0	4159	1664

Well Stickup = 2 ft
Ground Elevation = ft

DH-8 DOWNHOLE SEISMIC MEASUREMENTS
CAMANCHE DAM



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DOWNHOLE SEISMIC MEASUREMENTS
MAIN DAM EMBANKMENT - DH-8
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure E-11

Rev. 3 02/05/10

Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer
0	0	1	0	1
3.3	1	1	2.5	1
8.3	1.9	1	3	
13.3	2.2	1	5	1
18.3	3.8	1	6	1
23.3	4.5	1	7	1
28.3	5.7	1	8.3	1
33.3	6.3	1	10	
38.3	7	1	11	1
43.3	7.1	1	13	1
48.3	8.7	1	15	1
53.3	9.4	1		
58.3	9.9	1		
63.3	11.7	1	18	1
68.3	12.4	1	19	1
78.3	14	1	23	1
87.3	16.9	1	25.5	1

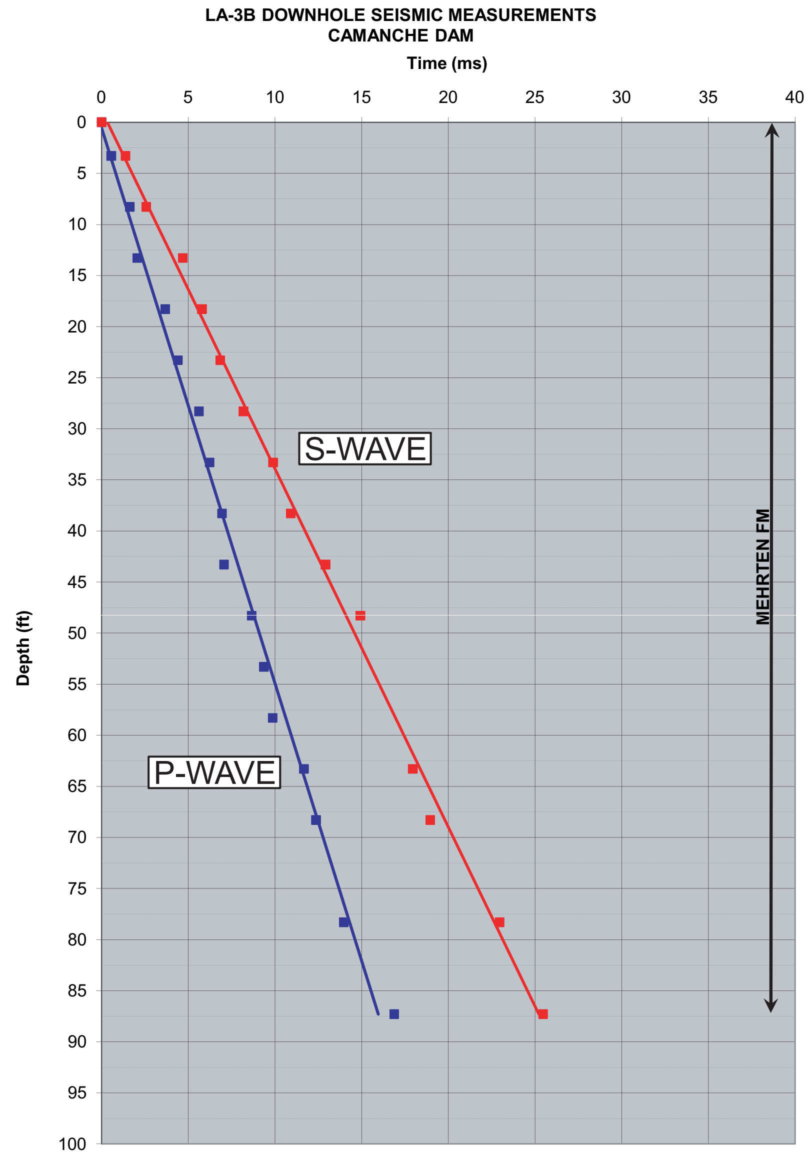
Offset (ft)
5

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	5435	0 to 87	3508	0 to 87
2				
3				
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 3.3	5991	2396
3.3 to 8.3	4644	4192
8.3 to 13.3	11580	2369
13.3 to 18.3	3113	4514
18.3 to 23.3	6810	4733
23.3 to 28.3	4121	3762
28.3 to 33.3	8102	2914
33.3 to 38.3	7033	4910
38.3 to 43.3	44631	2492
43.3 to 48.3	3124	2492
48.3 to 53.3	7091	
53.3 to 58.3	9903	
58.3 to 63.3	2778	
63.3 to 68.3	7110	4974
68.3 to 78.3	6232	2498
78.3 to 87.3	3103	3593

Well Stickup = 1.7 ft
Ground Elevation = 223 ft



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DOWNHOLE SEISMIC MEASUREMENTS
MAIN DAM - LEFT ABUTMENT - LA-3B
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-12

Rev. 3 02/05/10

Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	5
5	3.4	12	5.5	1	
10	4.2	2	7.5	2	
15			9	2	
20	5.3	2	10.5	2	
25	6.2	2			
30	6.8	2	14	2	
35	8.2	2			
40	8.9	2	16	2	
45	10	2	18	2	
50	10.9	2	20	2	
55	12.4	2			
60	14.3	2	22	2	
65					
70	15.7	2	26	2	

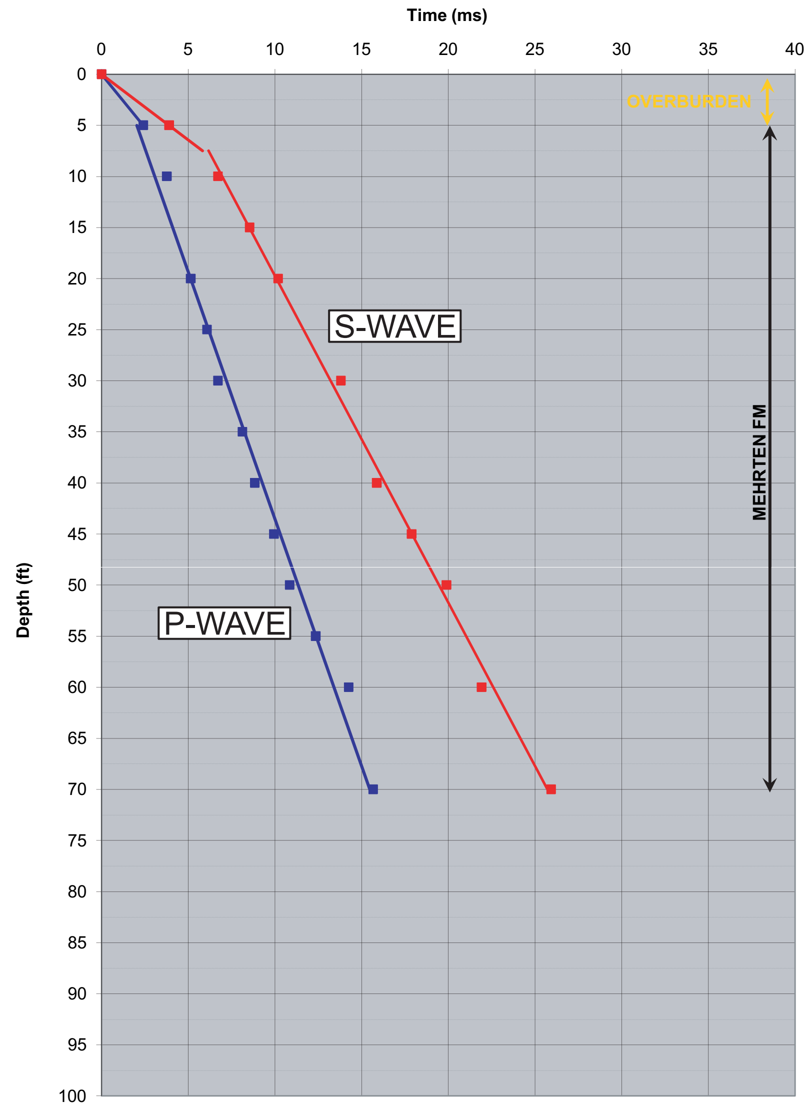
Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	2080	0 to 5	1286	0 to 8
2	4840	5 to 70	3196	8 to 70
3				
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 5	2080	1286
5 to 10	3697	1774
10 to 15		2732
15 to 20		3033
20 to 25	5331	
25 to 30	7963	
30 to 35	3546	
35 to 40	7006	
40 to 45	4514	2483
45 to 50	5512	2487
50 to 55	3326	
55 to 60	2629	
60 to 65		
65 to 70		

Well Stickup = 0 ft
Ground Elevation = 220 ft

RA-3B DOWNHOLE SEISMIC MEASUREMENTS
CAMANCHE DAM



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DOWNHOLE SEISMIC MEASUREMENTS
MAIN DAM - RIGHT ABUTMENT - RA -3B
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-13

Rev. 3 02/05/10

Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	5
4	4	1	5	1	
9	4.2	1	8.5	1	
14	6.5	1	13	1	
19	8	1	18.5	1	
24	12.5	1	22.2	1	
29	14.9	1	26	1	
34	17	1	30.5	1	
39	19.9	1	34	1	
44	21.9	2	37.5	1	
49	23.7	2	41.5	1	
54	25.4	2	46	1	
59	27.1	2	50	1	
64	28.8	23	55	1	
69	30	3	59	1	
74	31	3	64.5	1	
79	32	3	69	1	
84	32.9	3	73	1	
89	34	3	77	1	
94	35	34	81	12	
99	35.3	4	83.5	2	
104	36.3	4	86	2	
109	36.9	4	87.5	2	

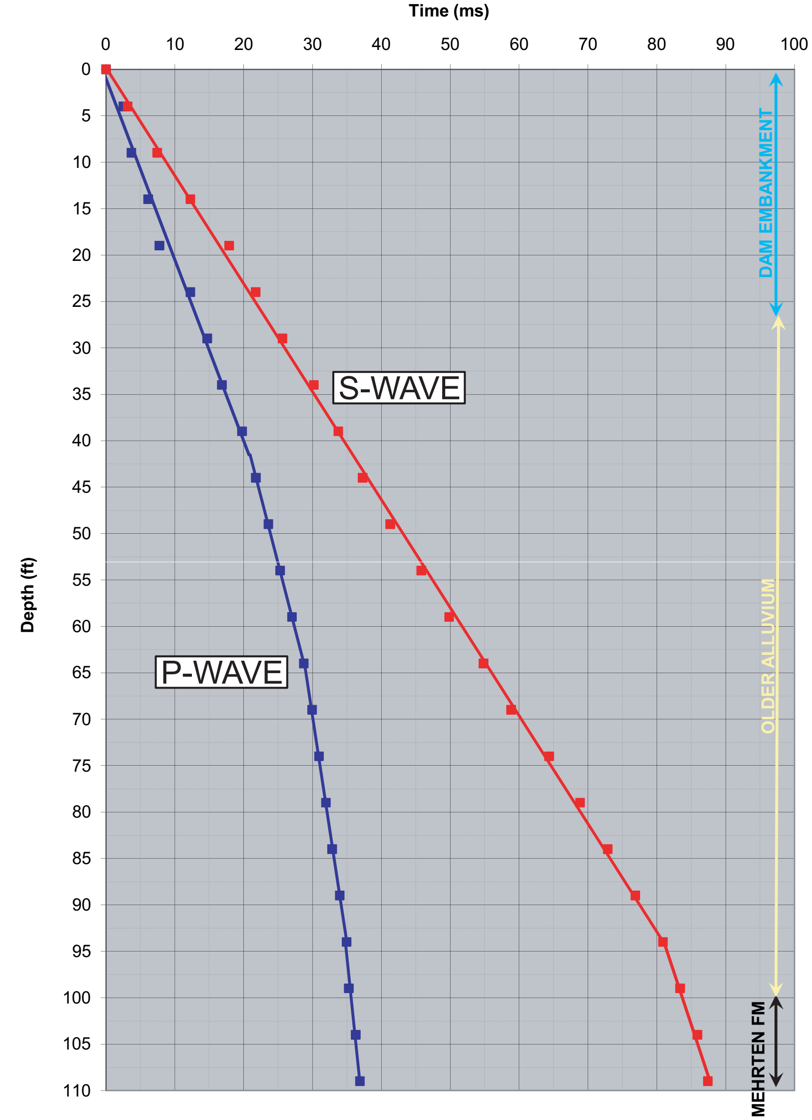
Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	1950	0 to 42	1163	0 to 94
2	2885	42 to 64	2237	94 to 109
3	4877	64 to 94		
4	7163	94 to 109		
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 4	1601	1281
4 to 9	4264	1161
9 to 14	2041	1039
14 to 19	3095	885
19 to 24	1111	1301
24 to 29	2044	1286
29 to 34	2341	1098
34 to 39	1713	1409
39 to 44	2473	1414
44 to 49	2751	1242
49 to 54	2917	1107
54 to 59	2922	1245
59 to 64	2925	998
64 to 69	4136	1246
69 to 74	4961	908
74 to 79	4967	1109
79 to 84	5520	1247
84 to 89	4527	1248
89 to 94	4979	1248
94 to 99	16422	1994
99 to 104	4985	1994
104 to 109	8290	3317

Well Stickup = 1 ft
Ground Elevation = 214.8 ft

BH-5 DOWNHOLE SEISMIC MEASUREMENTS
CAMANCHE DAM



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DOWNHOLE SEISMIC MEASUREMENTS
DIKE 2 - BH-5
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-14

Rev. 3 02/05/10

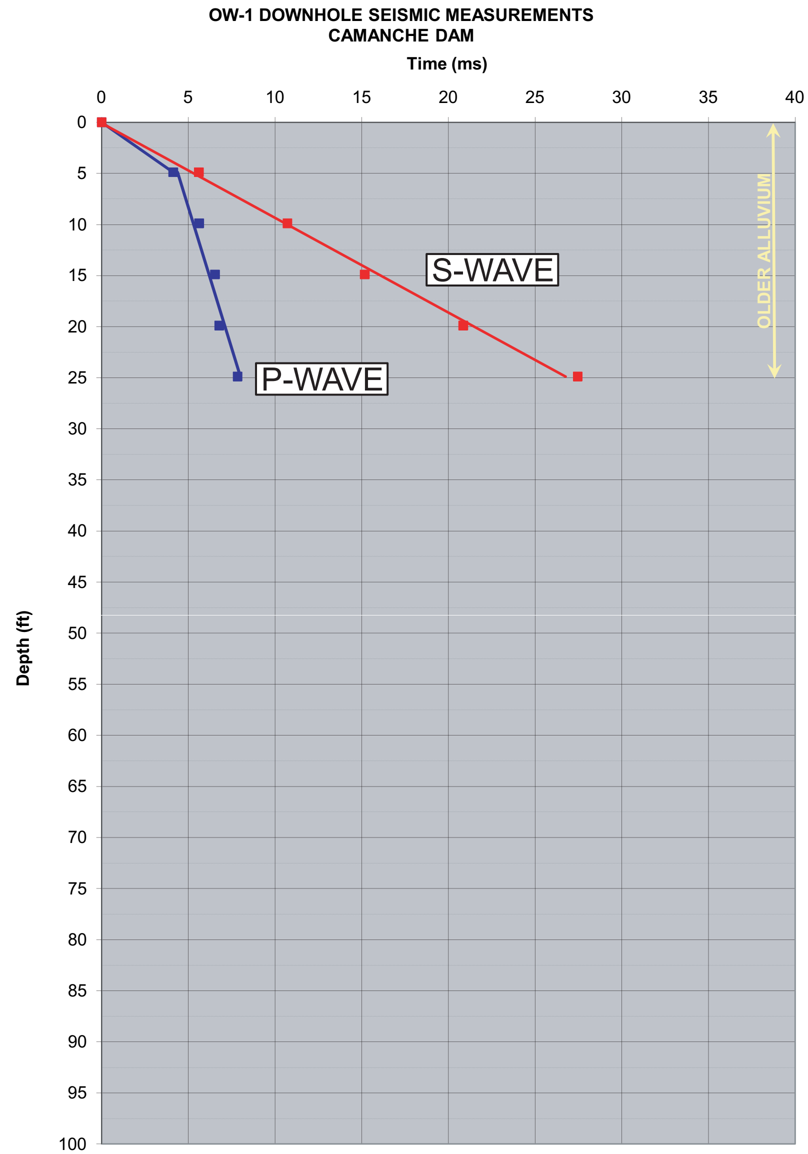
Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	5
4.9	5.9	12	8	1	
9.9	6.3	2	12	1	
14.9	6.9	2	16	1	
19.9	7	2	21.5	1	
24.9	8	2	28	1	

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	1187	0 to 5	928	0 to 25
2	5521	5 to 25		
3				
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 4.9	1187	875
4.9 to 9.9	3347	978
9.9 to 14.9	5446	1122
14.9 to 19.9	20204	880
19.9 to 24.9	4742	758

Well Stickup = 0.1 ft
Ground Elevation = 183.5 ft



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DOWNHOLE SEISMIC MEASUREMENTS
DIKE 2 - OW-1
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure E-15

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Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	5
3.6	5.9	12	9	12	
8.6	6.4	2	12	2	
13.6	6.9	2	16	2	
18.6	7	2	19.5	2	
23.6	7.1	2	23	2	
28.6	7.5	2	27.5	2	
33.6	8	2	32.5	23	
38.6	9	2	38.5	3	
43.6	9.5	2	44.5	3	

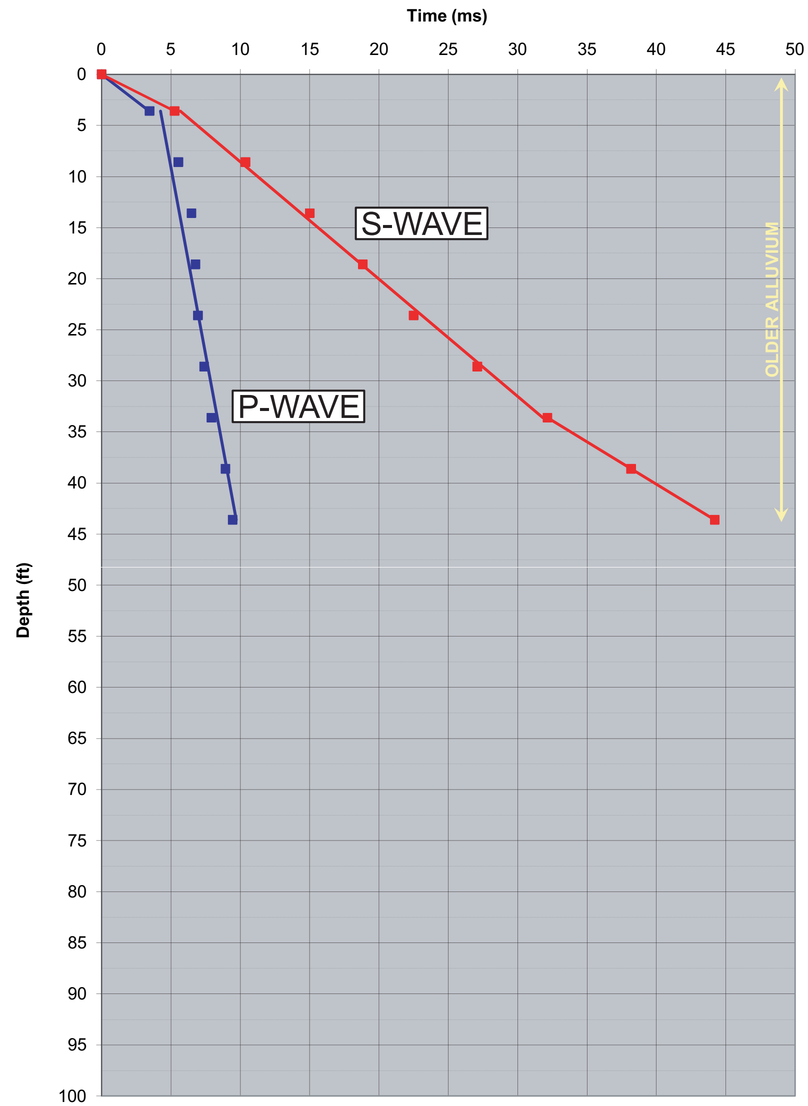
Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	1044	0 to 4	685	0 to 4
2	7312	4 to 44	1149	4 to 34
3			829	34 to 44
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 3.6	1044	685
3.6 to 8.6	2398	977
8.6 to 13.6	5300	1077
13.6 to 18.6	17617	1311
18.6 to 23.6	26909	1363
23.6 to 28.6	11309	1090
28.6 to 33.6	9525	989
33.6 to 38.6	4938	829
38.6 to 43.6	9752	829

Well Stickup = 1.4 ft
Ground Elevation = 189.5 ft

OW-4 DOWNHOLE SEISMIC MEASUREMENTS
CAMANCHE DAM



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DOWNHOLE SEISMIC MEASUREMENTS
DIKE 2 - OW-4
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-16

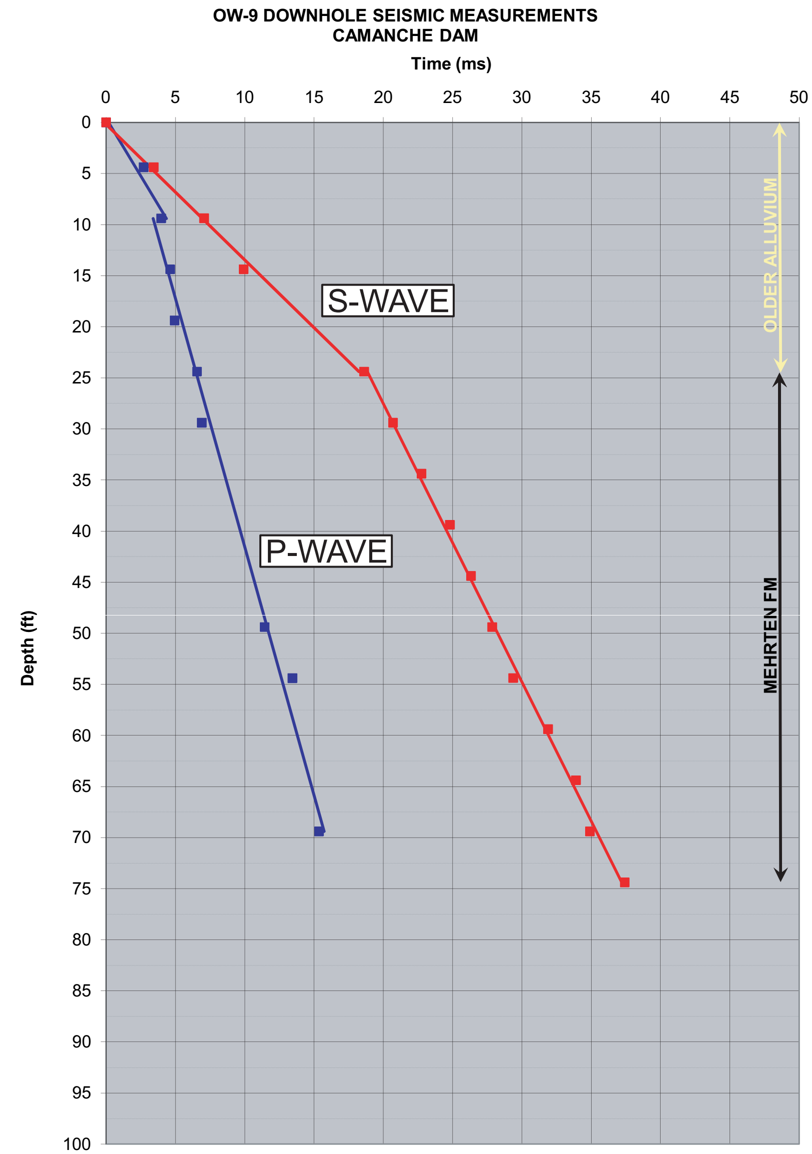
Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	5
4.4	4.1	1	5.2	1	
9.4	4.5	12	8	1	
14.4	4.9	2	10.5	1	
19.4	5.1	2			
24.4	6.7	2	19	12	
29.4	7	2	21	2	
34.4			23	2	
39.4			25	2	
44.4			26.5	2	
49.4	11.5	2	28	2	
54.4	13.5	2	29.5	2	
59.4			32	2	
64.4			34	2	
69.4	15.4	2	35	2	
74.4			37.5	2	

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	2249	0 to 9	1327	0 to 24
2	4861	9 to 69	2723	24 to 74
3				
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 4.4	1624	1281
4.4 to 9.4	3955	1378
9.4 to 14.4	7622	1751
14.4 to 19.4	16144	
19.4 to 24.4	3077	
24.4 to 29.4	14823	2393
29.4 to 34.4		2429
34.4 to 39.4		2451
39.4 to 44.4		3263
44.4 to 49.4		3281
49.4 to 54.4	2498	3293
54.4 to 59.4		1991
59.4 to 64.4		2487
64.4 to 69.4		4943
69.4 to 74.4		1995

Well Stickup = 0.6 ft
Ground Elevation = 188 ft



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DOWNHOLE SEISMIC MEASUREMENTS
DIKE 2 - OW-9
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-17

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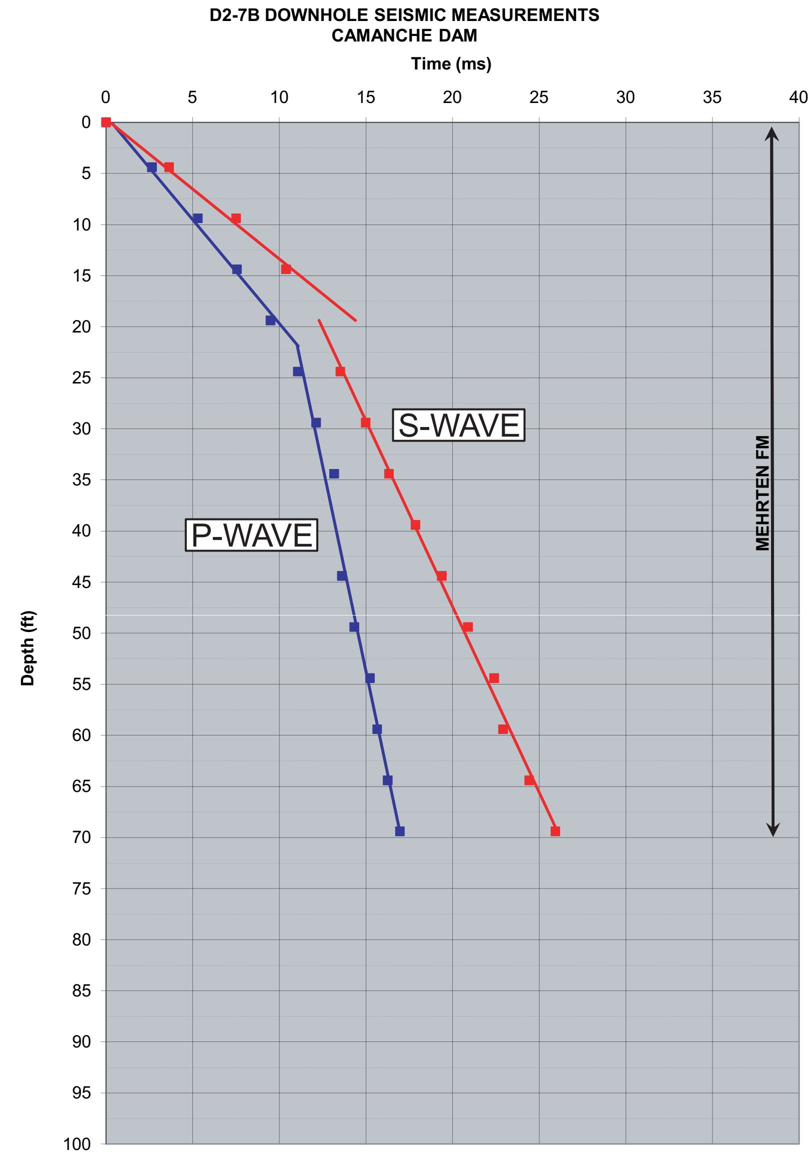
Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	5
4.4	4	1	5.5	1	
9.4	6	1	8.5	1	
14.4	8	1	11	1	
19.4	9.8	1			
24.4	11.3	2	13.8	2	
29.4	12.3	2	15.2	2	
34.4	13.3	2	16.5	2	
39.4			18	2	
44.4	13.7	2	19.5	2	
49.4	14.4	2	21	2	
54.4	15.3	2	22.5	2	
59.4	15.7	2	23	2	
64.4	16.3	2	24.5	2	
69.4	17	2	26	2	

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	2031	0 to 22	1369	0 to 19
2	8034	22 to 69	3636	19 to 69
3				
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 4.4	1665	1211
4.4 to 9.4	1883	1292
9.4 to 14.4	2212	1732
14.4 to 19.4	2587	
19.4 to 24.4	3164	
24.4 to 29.4	4735	3411
29.4 to 34.4	4827	3721
34.4 to 39.4		3271
39.4 to 44.4		3288
44.4 to 49.4	7014	3299
49.4 to 54.4	5501	3306
54.4 to 59.4	12228	9739
59.4 to 64.4	8245	3317
64.4 to 69.4	7093	3319

Well Stickup = 0.6 ft
Ground Elevation = 211 ft



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DOWNHOLE SEISMIC MEASUREMENTS
DIKE 2 - D2-7B
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-18

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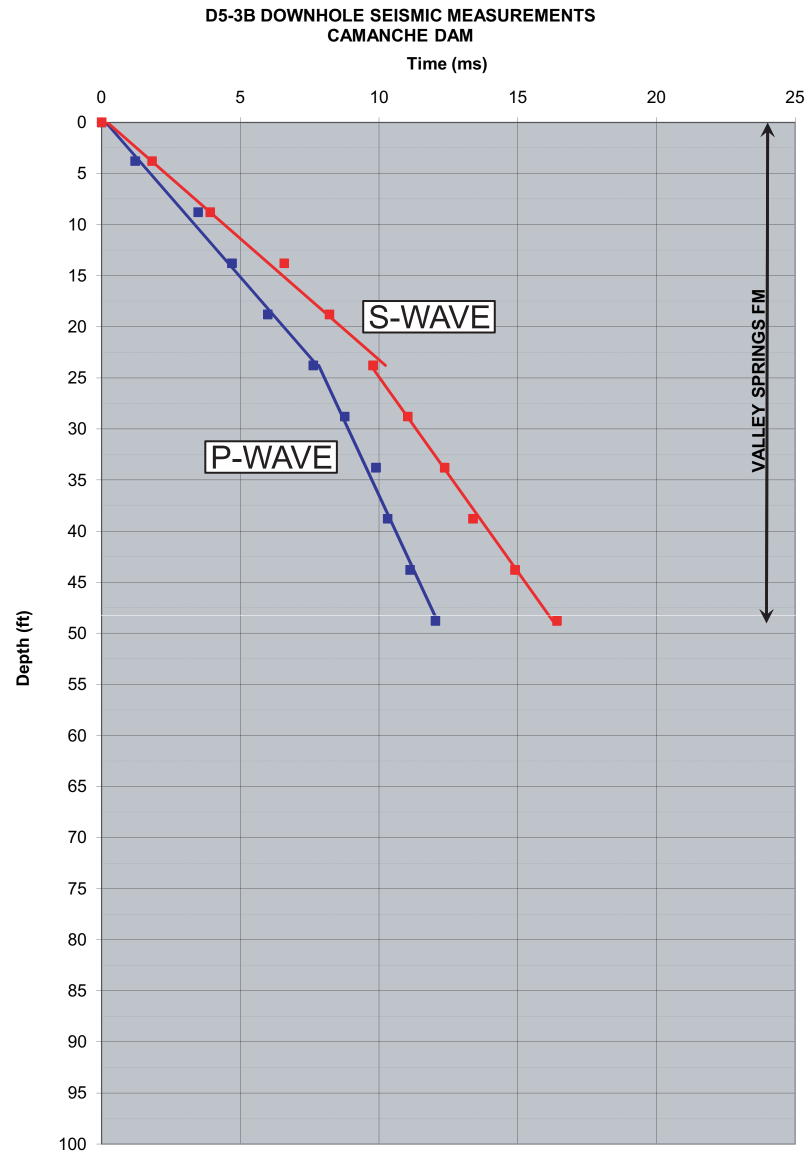
Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer	Offset (ft)
0	0	1	0	1	5
3.8	2	1	3	1	
8.8	4	1	4.5	1	
13.8	5	1	7	1	
18.8	6.2	1	8.5	1	
23.8	7.8	12	10	12	
28.8	8.9	2	11.2	2	
33.8	10	2	12.5	2	
38.8	10.4	2	13.5	2	
43.8	11.2	2	15	2	
48.8	12.1	2	16.5	2	

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	3122	0 to 24	2372	0 to 24
2	5838	24 to 49	3811	24 to 49
3				
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 3.8	3140	2093
3.8 to 8.8	2205	2384
8.8 to 13.8	4088	1874
13.8 to 18.8	3874	3062
18.8 to 23.8	3046	3181
23.8 to 28.8	4403	4005
28.8 to 33.8	4450	3758
33.8 to 38.8	11838	4884
38.8 to 43.8	6150	3303
43.8 to 48.8	5499	3309

Well Stickup = 1.2 ft
Ground Elevation = 222 ft



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DOWNHOLE SEISMIC MEASUREMENTS
DIKE 5 - D5-3B
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-19

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Depth (ft)	P-time (ms)	P-layer	S-wave (ms)	S-layer
0	0	1	0	1
3.4	1.5	1	3.8	1
8.4	2	1	4	1
13.4	2.8	1	6.5	1
18.4	3.1	1	7.8	1
23.4	4.8	1	9.5	1
25.4	5.3	1	10.5	1

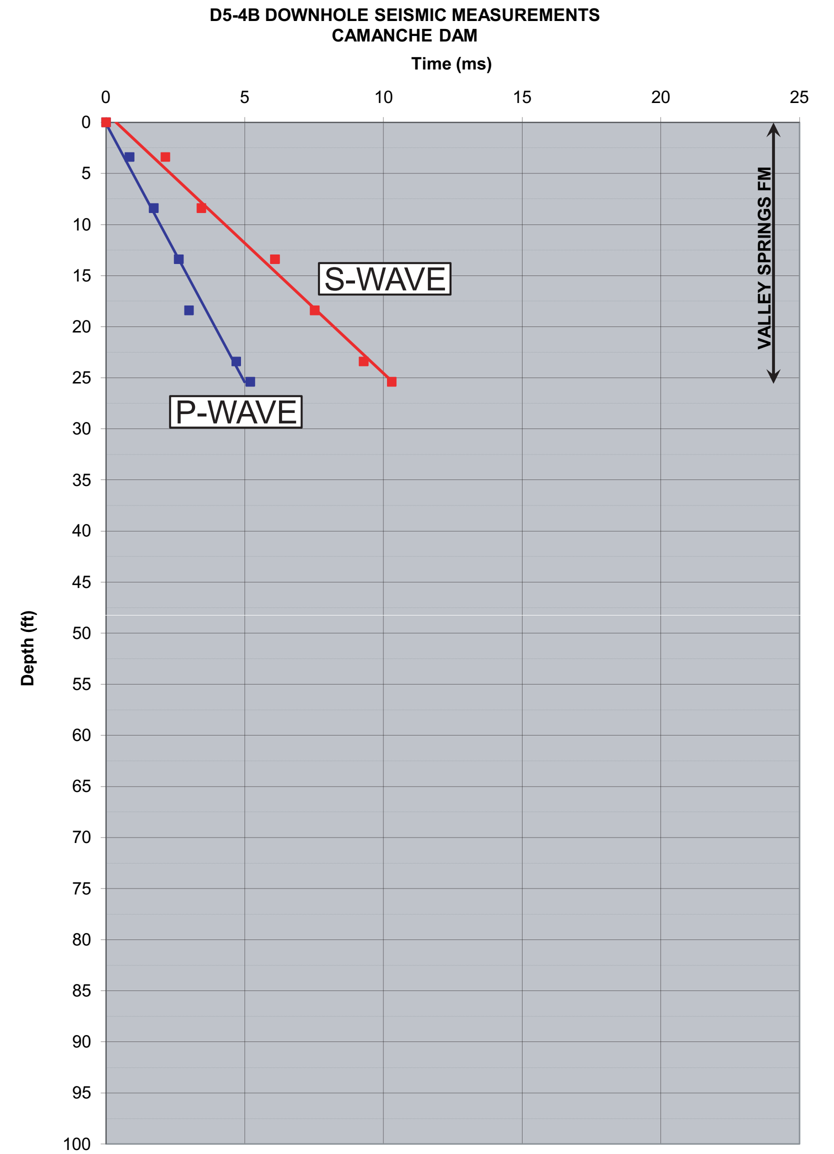
Offset (ft)
5

Layer	P-Velocity (fps)	P-Depth (ft)	S-Velocity (fps)	S-Depth (ft)
1	5059	0 to 25	2547	0 to 25
2				
3				
4				
5				
6				
7				
8				
9				
10				

INTERVAL VELOCITES

Depth Range (ft)	P-Velocity (fps)	S-Velocity (fps)
0 to 3.4	4031	1591
3.4 to 8.4	5713	3845
8.4 to 13.4	5526	1885
13.4 to 18.4	13580	3479
18.4 to 23.4	2937	2836
23.4 to 25.4	3951	1976

Well Stickup = 1.6 ft
Ground Elevation = 235.5 ft



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DOWNHOLE SEISMIC MEASUREMENTS
DIKE 5 - D5-4B
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
E-20

Rev. 3 02/05/10

APPENDIX F
REVIEW OF REGIONAL SEISMICITY

INTRODUCTION

The discussion on project area seismicity presented in Section 5.2.2 of the Supporting Technical Information Document (STID) (EBMUD and GEI, 2008) suggests that the controlling earthquake (magnitude and source) as recommended by Wahler in 1981 has not changed and is still valid for the present analyses. That section states: "Since 1983, new attenuation relationships have been developed and additional geologic studies of the Foothills fault system have been completed. However, the general earthquake magnitudes (M 6.5) and site to source distances used by Wahler have not changed significantly." Table 5.1 from the STID indicates that the Ione and Waters Peak faults (Bear Mountains Fault zone), at distances of 15 km (9 miles), constitute the controlling earthquake sources for Camanche Reservoir.

Following our initial seismicity data review, it was concluded that some further examination of the assumptions regarding controlling earthquake sources was warranted. For example, air photo lineament mapping along the Foothills Fault system, originally conducted by Woodward Clyde Consultants (WCC) in 1977 and recently presented by Page and Sawyer (2004), shows a number of mostly northwest-trending lineaments west of the Bear Mountains Fault zone and closer to Camanche Reservoir than the assumed controlling source along the Ione Fault. Additionally, detailed geologic mapping of Cenozoic deposits in the project region by the USGS (Marchand and Bartow, 1979) shows several post-Mehrten faults approximately 4.5 miles northwest of the Main Dam. Although the potential for local earthquakes and fault rupture hazard at the reservoir were briefly discussed by Wahler in their 1981 seismic re-evaluation of the Main Dam, neither the WCC lineaments nor the USGS mapping were addressed in the recent STID.

The present review is therefore intended to re-examine potential earthquake sources for Camanche Reservoir. The review is based on photolineament examination using several sets of air photo imagery, review of earlier faulting and lineament evaluations conducted for several nearby projects (Raised Pardee Dam, ESA, 1992; New Hogan Dam, US Army Corps of Engineers, 1995; Landsat and U-2 Photolineament Maps, USGS, 1979; Camanche Seismic Re-evaluation, Wahler, 1981), and review of USGS and CDMG geologic maps of the region. Lineaments identified during the present review and during previous studies were compiled and presented on several figures included herein and discussed in the following sections.

DATA REVIEW

The following data was reviewed as part of the seismicity review:

- Small scale (1:125,000) NASA U2 false color IR stereo photo imagery of the Foothills Fault zone area extending to north of Jackson Creek Reservoir and Ione, and to south of New Hogan Reservoir, and west to approximately the central portion of Camanche Reservoir;
- Intermediate scale (~1:42,000) black and white stereo photos of the project area across Camanche and Pardee reservoirs and to just north of New Hogan (provided by the District);
- Google earth imagery;
- CDMG/Wagner, 1981 Geologic Map of the Sacramento Quadrangle;
- Earth Sciences Associates, 1992, Raised Pardee Dam Geotechnical Investigation Vol. 1;

- GEI Consultants, 2008, Camanche Dam Supporting Technical Information Document and Sixth Five-Year Part 12D Safety Inspection Report;
- HCG, 1998, Pardee Reservoir Enlargement Project, Preliminary Design Report, Volume 6, Seismotectonic Evaluation Study;
- Hodges, 1979, Preliminary Maps of Photolineaments along Parts of the Western Sierra Nevada Foothills, Based on Landsat Images and U2 Aircraft Photographs, USGS OFR 79-1470;
- Jennings, 1994 Fault Activity Map of California;
- Marchand and Bartow, 1979, Preliminary Geologic Map of Sutter Creek Quadrangle, USGS OFR 79-436;
- Page and Sawyer, 2004, Overview of Late Cenozoic Faulting in the Sierra Nevada Foothills; includes lineament mapping originally presented in Woodward-Clyde Consulatnts (WCC) 1977 report on faulting and seismicity in the northern Sierra Nevada region of PG&E dams;
- PG&E and others, 2007 (October), Regional Geology, Seismicity, and General Ground Motion Considerations for the Merced, San Joaquin and Kings Hydroelectric Systems;
- U.S. Army Corps of Engineers, 1995, New Hogan Dam Geologic and Seismologic Investigation; includes geomorphic profile just north of Camanche, and lineament map;
- Wahler Associates, 1981, Seismic Re-Evaluation Camanche Reservoir Main Dam.

PROCEDURE

This seismicity review included the following steps:

- Superimposing and plotting the WCC/Page and Sawyer lineaments onto a copy of the USGS Landsat photolineament map (see Figure F-1);
- Comparison of these lineaments with geologic mapping of the area by Marchand and Bartow, and CDMG/Wagner;
- Air photo examination of the B&W and false color IR stereo photo sets, and plotting observed lineaments onto a 1:100,000 scale topographic base (see Figure F-2); the Page and Sawyer/WCC lineaments were also transferred to this map;
- Review of seismicity evaluations including faulting and lineament mapping conducted for the New Hogan, Raised Pardee and Wahler seismic re-evaluation studies.

GENERAL SEISMIC SETTING

Camanche Reservoir is located within the westernmost foothills of the Sierra Nevada, at the east margin of the Central Valley. At this location, the Main Dam and westernmost dikes are situated at a distance of approximately 9 miles (15 km) from the nearest presently identified faults that comprise the Foothills Fault system.

At the latitude of Camanche, the Foothills Fault system consists of two relatively closely spaced zones of faulting, the Bear Mountains Fault zone on the west and the Melones Fault zone slightly further east, both shown on Figure F-3. Both zones include a number of discrete northwest-trending fault strands, a few of which display small amounts of late-Cenozoic, mostly east side-down displacement. The Melones and Bear Mountains zones include several faults suspected of post-Mehrten displacement (less than ~4 million years), and locally these faults were considered as potential seismic sources for Pardee Dam during the recent Pardee Reservoir Enlargement Project (HCG, 1998). These faults include:

- The Ione Fault (shown on Figures F-2 and F-3), which is located 9 miles (15 km) to the east of Camanche Reservoir and just west of the northern reach of Pardee Reservoir. Page and Sawyer (2004) show this fault as having evidence of movement during the latest Quaternary (i.e., within the last 15,000 years), although apparently this evidence is not absolutely unequivocal;
- The Waters Peak Fault (Figure F-3) at approximately the same distance from Camanche as the Ione Fault, and which passes immediately downstream and south of Pardee Dam;
- The Devils Gate fault (Figure F-3) that passes along the east shore of Pardee Reservoir;
- The Youngs Creek Fault (Figure F-3) is the easternmost fault within the Bear Mountains zone and is approximately 13 miles (21 km) east of Camanche. This fault is shown as a fault of undivided Quaternary age (movement within the last ~1.6 million years) on the Fault Activity Map of California (Jennings, 1994). Page and Sawyer show this as a suspected Quaternary-age fault with 5m east down displacement of the Mehrten;
- The Poorman Gulch Fault (Figure F-3) further to the east is the principal section of the Melones Fault (approximately 19 miles east of Camanche) that displays post-Mehrten displacement within this local region. The Poorman Gulch Fault is shown as a late Quaternary fault (movement within the last ~700,000 years) on the Fault Activity Map of California.

RESULTS

The seismicity review suggests the following:

- A strong, well-expressed northwest-trending lineament along the Ione Fault appears to be the closest pronounced linear geomorphic feature to Camanche Reservoir. This strong expression consists of what is predominately a west-facing escarpment situated just west of Lake Amador/Jackson Creek Dam and the north reach of Pardee Reservoir, and which extends from Highway 88 southward to its end at a point approximately 1.5 km north of the Mokelumne River, just south of Waters Peak as shown on Figure F-2. North of Highway 88 the Ione Fault appears to continue northwestward, with weaker expression (some aligned northwest-trending drainages and vegetation lineaments, and southwest-sloping topography), to a point approximately 5.5 km north of Ione and just south of Dry Creek. Between these north and south end-points as defined by the present review, its total length appears to be approximately 17 km. For portions of its length, along its southern and northern-most extents, the Ione Fault appears to closely follow the northwest-trending bedrock contact of Gopher Ridge volcanics on the west with Salt Springs slate on the east.
- Further to the south, a weak lineament through a saddle within Tvs rocks (Valley Springs Formation), just to the west of Valley Springs and north of Highway 12, is roughly in line

with the well-expressed segment of the fault 7 km to the north (Figure F-2). ESA (1992) reported an exposure of shears and possibly offset volcanic rocks in the same general vicinity but apparently at a short distance to the northeast, at Valley Springs Peak. Short and weak lineaments recognized south of this point (i.e., just west of Valley Springs and south of Highway 12) appear to be within Mesozoic basement rocks.

- All of the lineaments on Page and Sawyer's map that are shown west of the geomorphically well-expressed Waters Peak Fault (along the west side of Pardee Reservoir) are indicated on that map as having only weak to moderate expression on air photo imagery. Similarly, on the "Geologic and Lineament Map (Plate 1)" from the U.S. Army Corps of Engineers' New Hogan Dam study (1995), all of the lineaments that are shown west of the Ione and Waters Peak lineaments are indicated as having only weak to moderate expression.
- A number of short, discontinuous lineaments (mostly northwest-trending and less than one mile in length) occur at distances that are closer to Camanche Reservoir than the well-pronounced Ione and Waters Peak faults (Figure F-2). Although not continuous over long distances, some of these short lineaments are aligned. One of the lines of aligned segments is 9-10 km east of the Main Dam and occurs along the same lineament defined as the "Chili Camp lineament" by ESA (1992); a comparison of these short linear segments with the longer and more continuous lineaments indicated by ESA is shown on Figure F-4. Although ESA indicates that this particular lineament and their adjacent "Section 22" lineament have "strong expression in Tertiary and younger rocks", evaluation of air photo imagery for the present review indicates that none of the observed linear segments along these trends appear to show any strong level of expression that is on the scale of that exhibited by the Ione and Waters Peak lineaments. Furthermore, the more well-expressed section of the ESA "Chili Camp" lineament (on the south side of the reservoir) passes through metamorphic basement rock (Salt Springs slate) rather than through "Tertiary and younger rocks", and the southern extension of their lineament does not pass through southwest-trending ridge spurs of Tvs (Valley Springs Formation) north of Highway 12, where it is shown to exist on their map.
- Short northwest-trending faults offsetting Mehrten and Valley Springs formations were mapped by Marchand and Bartow (1979) along the east side of Rabbit Creek immediately north of the reservoir, approximately 7 km northwest of the Main Dam. These supposed faults have no geomorphic expression.
- Most of the short lineaments recognized west of the Foothills Fault zone/Ione trace appear to be aligned along northwest or southeast-trending drainages, and many do not cross through ridges/topographic barriers (some of which are clearly remnant Tertiary volcanics) toward which they project, as shown on Figure F-2. Other lineaments further east and south, within the metamorphic belt, are probably aligned/eroded along the strong northwest-trending foliation/jointing fabric that characterizes those rocks (e.g., the prominent northwest-trending basement rock contact of Gopher Ridge volcanics and Salt Springs slate, which controls the linear alignment of Slate Creek between New Hogan and Salt Spring Valley reservoirs).
- Wahler (1981) also conducted a photolineament review of the local Camanche Reservoir area and recognized only very short, discontinuous northeast-trending features (these lineaments have similar orientation to the northeast-trending reaches of the Mokelumne River just downstream of the Main Dam and along Dry Creek to the north; as shown on Figure F-1,

"poorly-defined" lineaments were identified along these drainages on the USGS Landsat analysis). Some or most of the Wahler-identified features appear to be along the northeast-southwest oriented eroded margins of resistant Mehrten layers, that are parallel to the principal southwest-flowing streams that drain the lower section of the tilted Sierra block in this region.

CONCLUSIONS

If any of the lineaments recognized west of the Ione/Waters Peak faults do actually represent late Cenozoic faulting, their weak expression and short lengths suggest a very low rate of activity that is even less than the already very low slip-rate acknowledged for the defined, well expressed traces of the Foothills Fault system to the east. Based on the results of probabilisitic seismic hazard studies of background earthquakes in the Sierran microplate (Sierran block), PG&E selected a background earthquake with a 3,000-year return period for their Sierra Nevada dams (PG&E, 2004). Deaggregation of the probabilistic seismic hazard data for that particular return period earthquake results in a magnitude 5.8 earthquake at 15 km. This equivalent deterministic maximum background earthquake is appropriate for the region of Camanche Reservoir, which is also situated within the Sierran block.

HCG (1998) considered the Ione Fault as a potential seismic source in their studies for the Pardee Reservoir enlargement project. Their report indicates the fault dips steeply west, with normal and right-lateral movement. HCG estimated a maximum earthquake magnitude of 6.4 for the Ione Fault, based on their estimated length of 23 km and using the 1994 Wells and Coppersmith magnitude/length correlations. As previously mentioned, the present review measures the total length of the Ione fault at 17 km, less than that used in the HCG study. Assuming capability of the Ione Fault, its estimated MCE of magnitude 6.4 (as per HCG) at 15 km from Camanche Reservoir would produce ground motions greater than the maximum PG&E background earthquake described above, and thus would control the seismic hazard at Camanche.

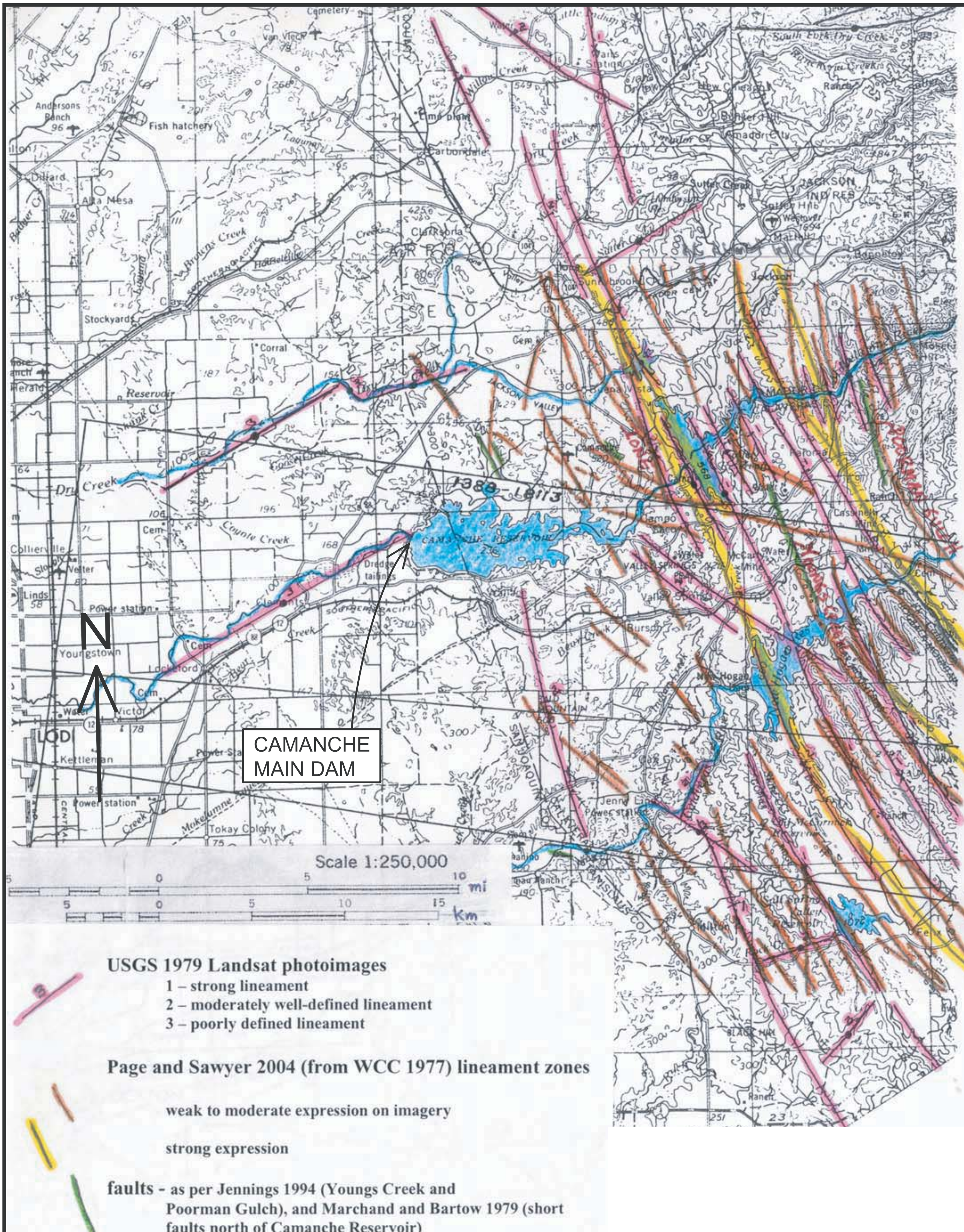
It should be noted that previous studies apparently have not unequivocally established evidence of active faulting (within the last 35,000 years) along either of the Ione or Waters Peak faults. For this reason the use of a M_w 6.5 earthquake on the Ione Fault at a distance on 15 km is considered conservative. It is also noted that this moment magnitude and distance easily accommodates the deterministic equivalent maximum background earthquake developed by PG&E for the Sierran block.

Fault parameters including degree of fault activity (i.e., slip rate) that should be incorporated in the ground motion evaluations are provided in the Table F-1. These parameters are based on the foregoing discussion and on information from other sources as indicated in the table.

TABLE F-1

SUMMARY OF FAULT PARAMETERS

Parameter Type	Parameter
Fault trace length at Endpoints	23km (HCG, 1998); 17 km (Terra/GeoPentech, present review)
Distance From Fault to Main Dam	15km
Moment Magnitude M_w	6.5
Fault Type	Normal/Oblique
Dip Angle and Direction of Fault	≥75 degrees west
Rupture Downdip width	12km (from CGS)
Hanging Wall or Footwall	N/A
Slip Rate of Fault	0.05 mm/yr
Shear Wave Velocity over top 30m	600 m/sec (see Appendix E)
Depth to 1 km/sec shear wave velocity	Unknown
Depth to 2.5 km/sec shear wave velocity	Unknown



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USGS LANDSAT AND WCC PHOTOLINEAMENT COMPILATION
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure F-1

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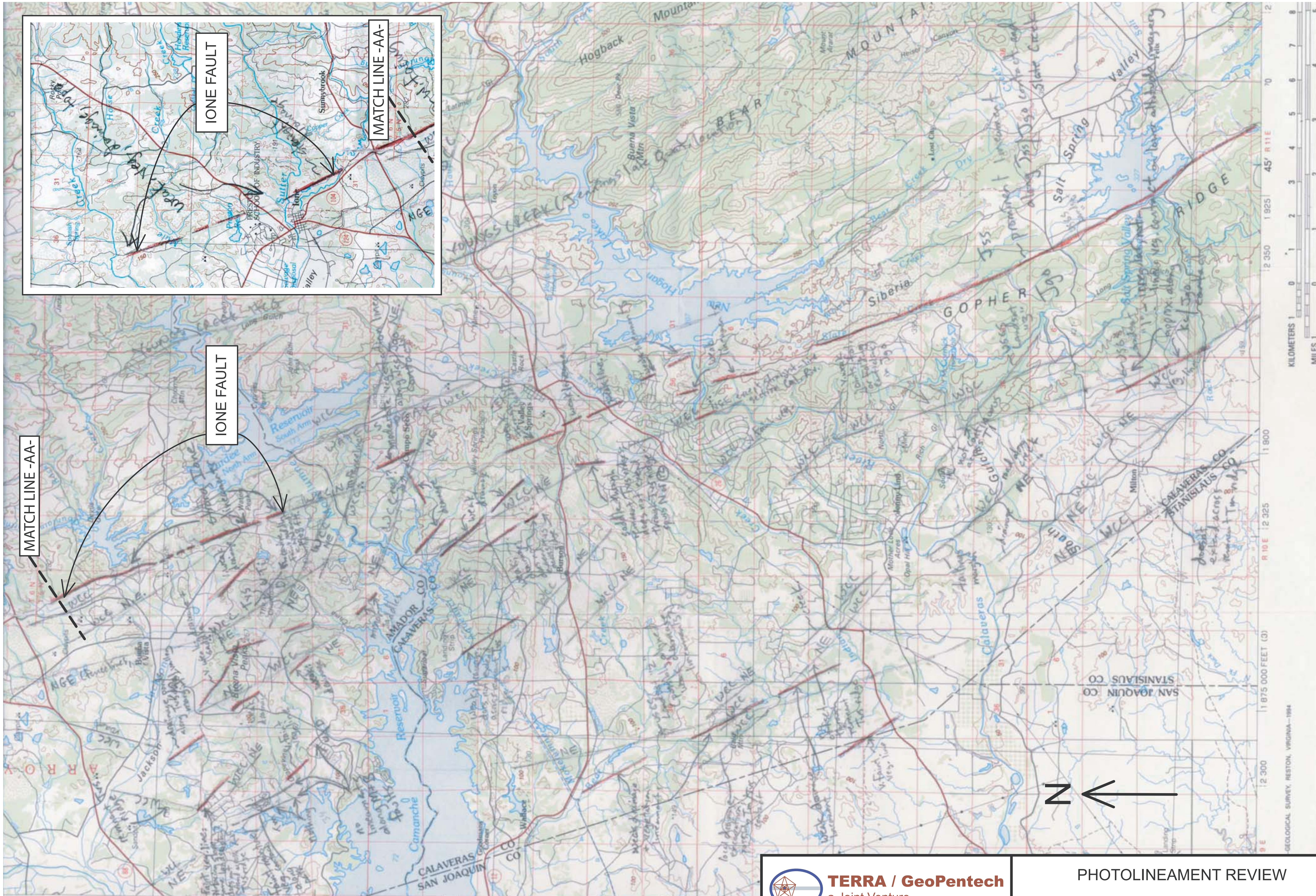

PHOTOLINEAMENTS RECOGNIZED DURING THIS REVIEW (TERRA / GEOPENTECH 2008)

PHOTOLINEAMENTS PER PAGE AND SAWYER (2004) AND WCC (1977)

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PHOTOLINEAMENT REVIEW

CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure F-2

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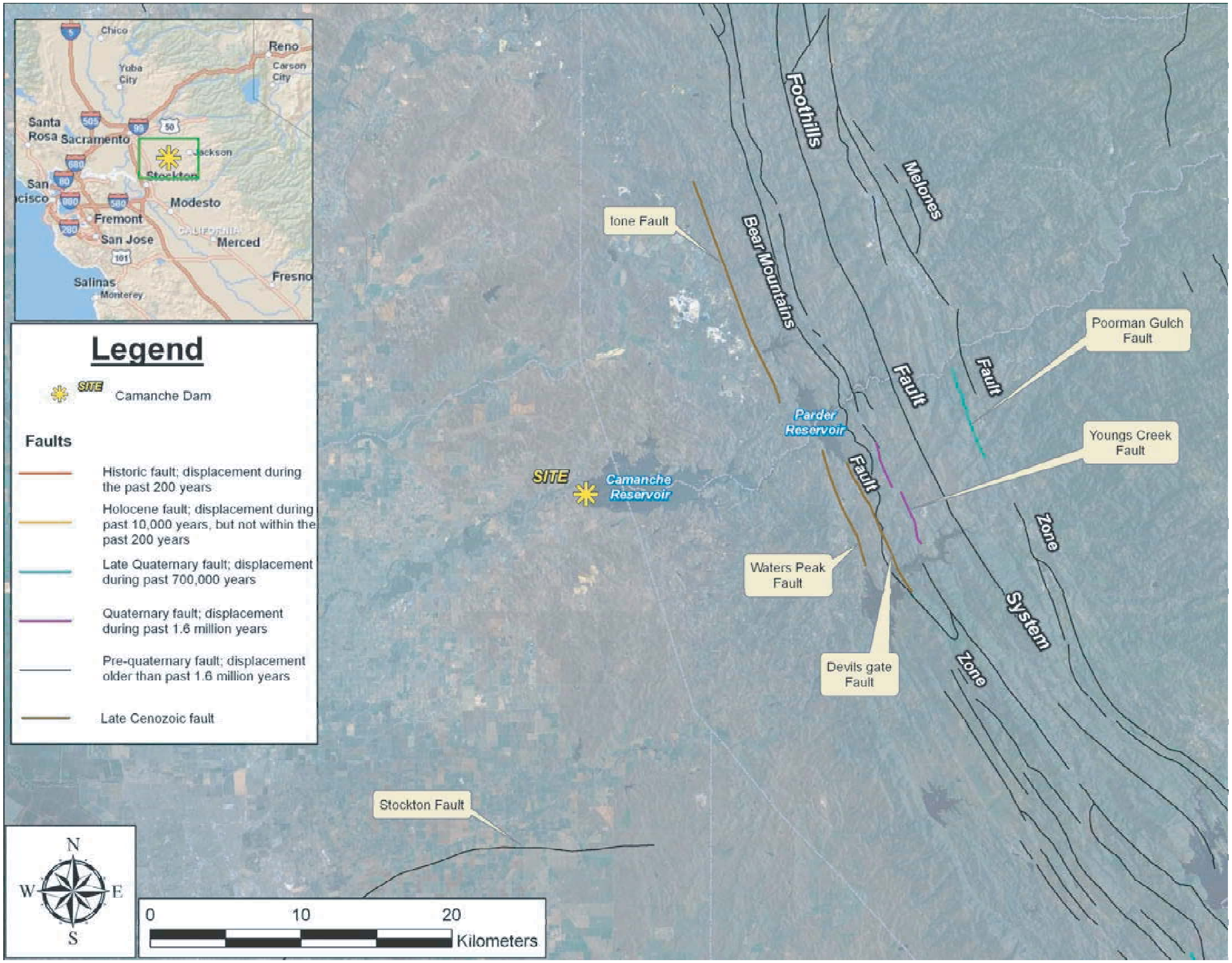

Note: Fault locations as per Jennings (1994) w/ exception of Ione fault; Ione fault location as per Terra engineers (2008) review.

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SEISMIC SOURCE MAP
MODIFIED FROM JENNINGS (1994)
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
F-3

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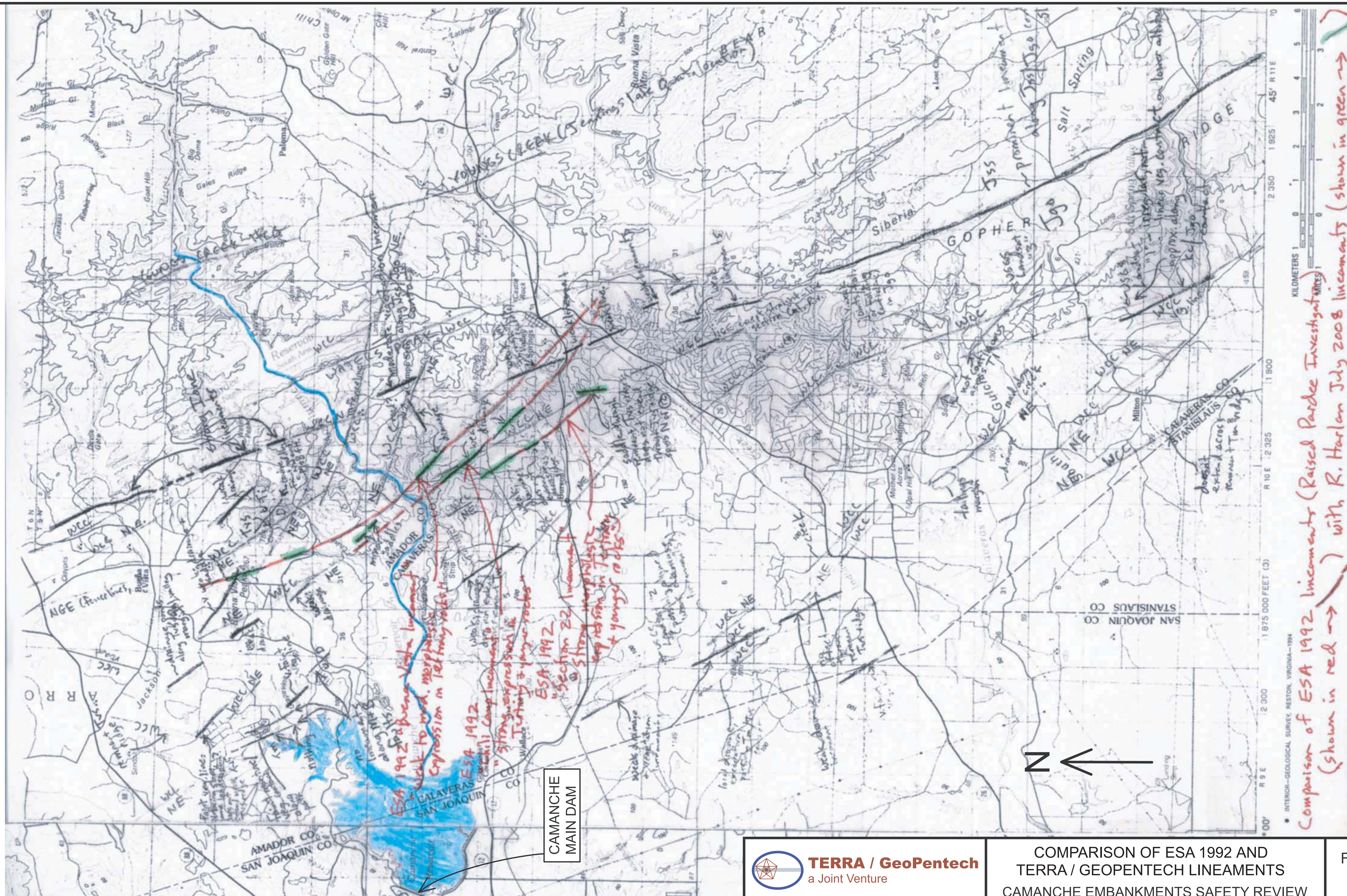

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COMPARISON OF ESA 1992 AND TERRA / GEOPENTECH LINEAMENTS

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Figure F-4

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APPENDIX G
BACKGROUND INFORMATION FOR SELECTION OF GROUND MOTIONS AND TIME HISTORIES

Enclosure 1
Shear Wave Velocity Profile Used in NGA Attenuation Relationships

The following documents our evaluation of the shear wave velocity profile of the foundation rock beneath the Main Dam. In general, the foundation beneath the Main Dam is composed of Tertiary-aged sedimentary rocks of the Mehrten, Valley Springs, and Ione formations, which are underlain by Cretaceous and Jurassic-aged basement rock of the Sierra Nevada bedrock complex.

The discussion of the shear-wave properties of the Main Dam foundation is divided into three parts: (1) approximate shear-wave velocity within the upper 30 m (100 ft) of foundation, (2) approximate depth to 1 km/sec (3,300 ft/s) foundation material, and (3) approximate depth to 2.5 km/sec (8,200 ft/s) foundation material.

Shear-Wave Velocity within the Upper 30 m (100 ft) of Foundation

The foundation within the first 30 m under the Main Dam consists of Mehrten and Valley Springs formation sedimentary rock. Based on data collected within 16 boreholes at the site (Appendix E), the shear-wave velocity within the upper 30 m of the foundation material ranges between approximately 0.64 km/sec (2,100 ft/sec) and 1.16 km/sec (3,800 ft/sec). The weighted average shear-wave velocity measured within this zone is about 0.76 km/s (2,500 ft/sec).

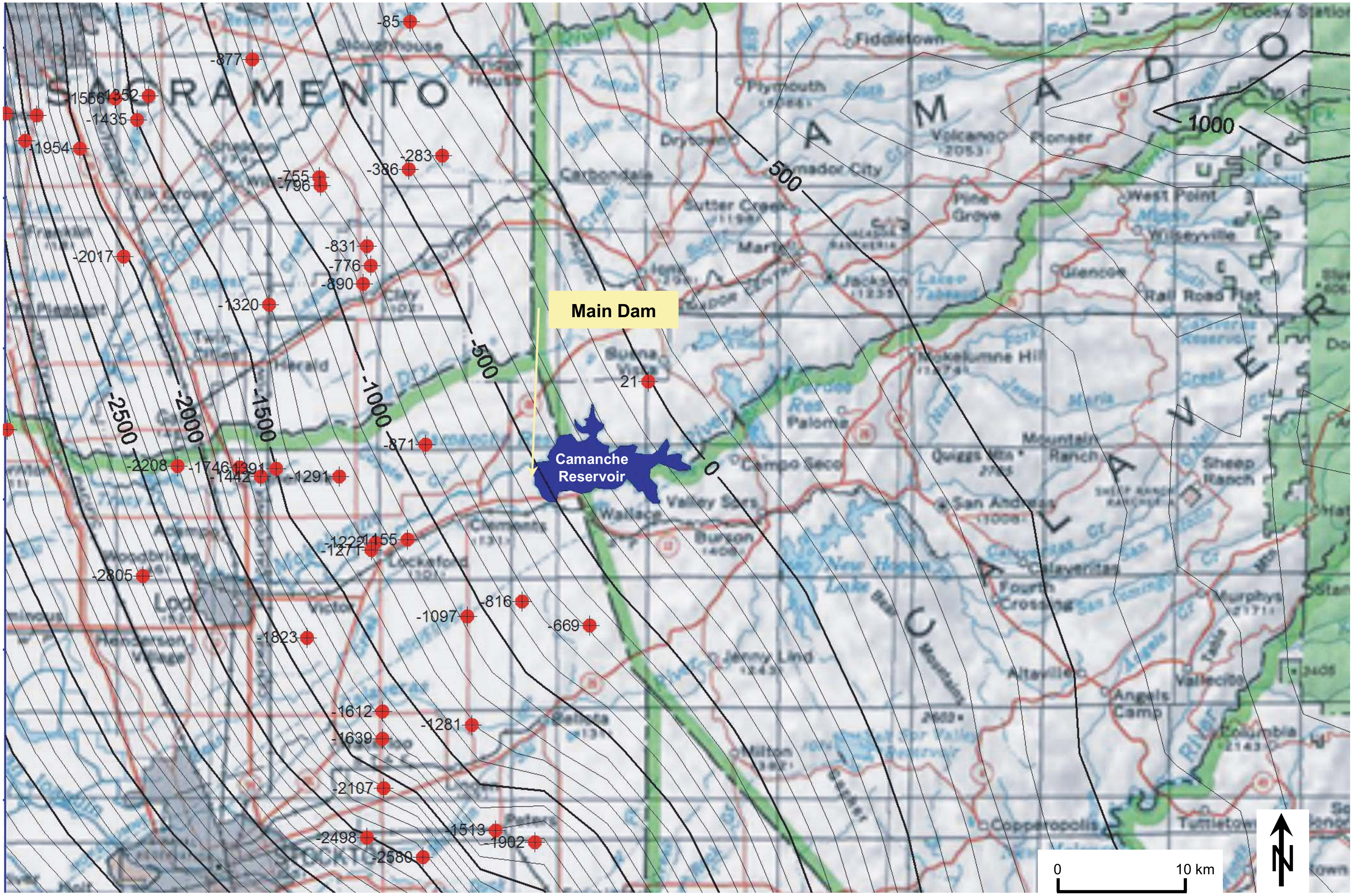
Approximate Depth to 1 km/sec (3,300 ft/s) Foundation Material

Based on the site specific shear-wave measurements by (Appendix E), the approximate depth to 1 km/sec rock beneath the Main Dam is estimated to be about of 30 m.

Approximate Depth to 2.5 km/sec (6,800 ft/s) Foundation Material

It is expected that the crystalline Sierra Nevada bedrock under the dam would have relatively high shear-wave velocities on the order of 2.5 km/sec. In order to estimate the depth to crystalline basement rock and therefore the depth to approximately 2.5 km/sec rock, deep wells near the site were reviewed (Wentworth, 1995). Figure G1-1 shows wells near the site that encountered basement rock with interpreted elevation contours on the top of the basement rock. As can be seen on Figure G1-1, the elevation to the top of the basement rock under the Main Dam is on the order of 550 m (1,800 ft) below sea level or about 600 m (2,000 ft) deep.

The approximate depth to 2.5 km/sec rock was also estimated based on shear-wave velocity versus depth relationships developed for different rock types in northern California by Brocher (2005). The relationships developed by Brocher for Sierran granitic rock estimate a shear-wave velocity of 2.5 km/sec at a depth of about 600 m (2,000 ft), which agrees with the depth estimate based on the depth to crystalline rock.



EXPLANATION

Contour on basement elevation in meters

-871 Well annotated with basement elevation (meters)

NOTE: Well data based on Wentworth et al. (1995)

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BASEMENT ROCK SURFACE CONTOURS

CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure G1-1

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Enclosure 2
Basis for Selection of
Three Adjusted Acceleration Time Histories

ENCLOSURE 2 SELECTION OF THREE ADJUSTED ACCELERATION TIME HISTORIES

INTRODUCTION

As discussed in Section 5 of the Report, the Maximum Design Earthquake ("MDE") ground motion is developed for the site in terms of response spectral values and three acceleration time histories adjusted to be compatible with the specified response spectral values. However, there were six candidate acceleration time histories used in this process: three originally proposed by the TERRA/GeoPentech team and three proposed by the California Division of Safety of Dams (DSOD). These six candidate acceleration time histories are listed in Table G2-1.

The following discussion provides the main reasons (a) why three of the six candidate adjusted acceleration time histories were not selected to represent the MDE ground motion, and (b) why the remaining three candidate adjusted acceleration time histories are considered more than adequate to represent the MDE ground motion.

PARAMETERS OF INTEREST IN EVALUATION

As can be seen in Table G2-1, the selected six candidate acceleration time histories as recorded (i.e., before adjustment) reflect usual parameters such as the earthquake magnitude and the distance to the fault rupture, which are in general consistent with the postulated earthquake conditions associated with the MDE ground motion. Apart from these usual parameters, the three parameters of particular interest in this evaluation are as follows:

- the Arias intensity;
- the 84th percentile spectral values associated with the MDE ground motion; and
- the 5% to 95% duration based on the plot of time versus Arias intensity.

Two Arias intensity attenuation relationships that are based on the NGA database are currently available: one by Travasarou, Bray and Abrahamson (2002), hereafter referred to as the Travasarou relationship, and the other by Watson-Lamprey and Abrahamson (2006), hereafter referred to as the Watson-Lamprey relationship. The Travasarou relationship basically provides the Arias intensity as a function of earthquake magnitude and distance from the seismic source; the Watson-Lamprey relationship, in addition to considering the earthquake magnitude and distance, provides the Arias intensity for a given shaking level as represented by the peak ground acceleration and the spectral value at a period of 1 second. The fact that the ground motion level is an important consideration in the Arias intensity values can be inferred from a standard deviation of 0.34 (natural log units) in the Watson-Lamprey relationship versus a standard deviation of 0.94 in the Travasarou relationship. Therefore, in evaluating the Arias intensity values of ground motions already selected to represent the 84^{th} percentile spectral values, the Watson-Lamprey relationship is considered more appropriate.

A "significant" duration of earthquake shaking is often defined as the time interval from 5% to 95% of the Arias intensity plotted as a function of time (Trifunac and Brady, 1975). A version of such duration relationship based on the NGA database has been published by Kempton and Stewart (2006). This Kempton duration relationship was also used in the evaluation. Note, however, that the Kempton duration relationship is similar to the Travasarou Arias intensity relationship in that the earthquake shaking level is not reflected in the relationship.

MDE GROUND MOTION

As provided in Table G2-1, the MDE ground motion corresponds to a moment magnitude of 6.5 at a distance of 13.5 km on a normal fault dipping 75 degrees west or toward the site. The median and 84th percentile Arias intensity values using the Watson-Lamprey relationship are 0.7 m/s and 1.0 m/s, respectively. The median and 84th percentile duration values using the Kempton duration relationship are 10.6 seconds and 16.5 seconds, respectively.

The MDE ground motion is represented by the 84th percentile response spectral values. Given this, the MDE acceleration time histories ideally should have a median Arias intensity value computed using relationships that reflect the fact that the time histories already represent 84th percentile response spectral values. The Watson-Lamprey relationship provides Arias intensity values given a level of shaking represented by spectral values at two periods. Therefore, an appropriate approach in developing MDE acceleration time histories would be to target their Arias intensity to be the median Arias intensity value using the Watson-Lamprey relationship, reflecting the effects of the MDE 84th percentile response spectral values. To select as the MDE time history an adjusted acceleration time history exhibiting, for example, an 84th percentile Arias intensity value given the MDE 84th percentile response spectral values (based on the Watson-Lamprey relationship) would mean selecting an MDE acceleration time history beyond the typical MDE 84th percentile response spectral values thus compounding conservatism.

As for the duration of the MDE time histories, given the nature of the duration relationship used, the target duration should be somewhere between 10.6 seconds (median) to 16.5 seconds (84th percentile).

CANDIDATE ACCELERATION TIME HISTORIES AS RECORDED

Table G2-1 lists the six candidate MDE acceleration time histories. The first three are those originally proposed by the TERRA/GeoPentech team; i.e., Loma Prieta, Northridge, and Kobe-Nishi-Akashi (Nishi-Akashi). The other three are those suggested by the DSOD; i.e., Imperial Valley-Cerro Prieto (Cerro Prieto); Morgan Hill; and Kobe-Kakogawa (Kakogawa).

The values of Arias intensity and duration provided in Table G2-1 are those of the acceleration time histories as recorded before any adjustments are made.

Figures G2-1A through G2-1F show the as-recorded acceleration time history, computed Arias intensity, and a comparison of the response spectral values of the as-recorded acceleration time history and the MDE spectral values for each of the six candidate time histories in the order provided in Table G2-1.

CANDIDATE ACCELERATION TIME HISTORIES AFTER ADJUSTMENTS

Table G2-2 lists the six candidate MDE acceleration time histories after they were adjusted to be compatible with the 84th percentile MDE response spectral values. Therefore, the values of Arias intensity, corresponding percentiles, and duration also provided in Table G2-2 are those for the adjusted acceleration time histories.

Figures G2-2A through G2-2F show the adjusted acceleration time history and computed Arias intensity of the adjusted acceleration time history for each of the six candidate time histories in the order provided in Table G2-1. Figures G2-2A through G2-2F also shown the computed median and 84th percentile Arias intensity values using the Travasarou relationship and the Watson-Lamprey relationship.

SELECTED ADJUSTED ACCELERATION TIME HISTORIES

Table G2-3A lists the three adjusted acceleration time histories selected to represent the MDE ground motion; i.e., Loma Prieta, Northridge, and Nishi-Akashi. Table G2-3B lists the three adjusted acceleration time histories that were not selected; i.e., Cerro Prieto, Morgan Hill, and Kakogawa.

Tables G2-3A and G2-3B also list the computed Arias intensity values and the corresponding percentile values (both using the Watson-Lamprey relationship), and the computed duration values (using the Kempton duration relationship).

DISCUSSION

The reasons for selecting the three adjusted acceleration time histories listed in Table G2-3A are discussed herein. The Arias intensity values of the six adjusted acceleration time histories are listed in the table below in order of decreasing computed Arias intensity values (median and 84th percentile Arias intensity values associated with the MDE ground motion are listed immediately beneath the table).

No.	Adjusted Time History	Arias Intensity (Watson-Lamprey relationship)	
		Value (m/s)	Percentile
1	Cerro Prieto*	2.8	100.0
2	Kakogawa*	1.4	98.5
3	Loma Prieta*	1.4	97.8
4	Nishi-Akashi	0.7	59.9
5	Northridge	0.7	46.8
6	Morgan Hill*	0.4	4.6

MDE Ground Motion: Arias intensity – median = 0.7 m/s; 84th percentile = 1.0 m/s

Given adjusted acceleration time histories representing the 84th percentile spectral values of the MDE ground motion, one would ideally select three acceleration time histories among all those with the Arias intensity value given the 84th percentile response spectral values being near median; otherwise, one would be introducing an additional conservative bias. In this sense the Cerro Prieto, Kakogawa, and Loma Prieta are outliers on the high end of the Arias intensity values (about 98 to 100 percentile values), and the Morgan Hill is an outlier on the low end. These outliers are identified by (*) in the above table. The very high Arias intensity value of the adjusted Cerro Prieto motion is particularly noteworthy.

The duration values of the six adjusted acceleration time histories are listed in the table below in order of decreasing computed duration values (median and 84th percentile duration values associated with the MDE ground motion are listed immediately beneath the table).

No.	Adjusted Time Histor	Duration (seconds) using the Kempton relationship
1	Cerro Prieto*	39.6
2	Kakogawa*	18.2
3	Northridge	13.6
4	Nishi-Akashi	12.3
5	Loma Prieta	11.0
6	Morgan Hill*	4.6

MDE Ground Motion: median = 10.6 seconds; 84th percentile = 16.5 seconds

Given adjusted acceleration time histories representing the 84th percentile spectral values of the MDE ground motion, one would ideally select three acceleration time histories with duration given the 84th percentile spectral values of the MDE ground motion being near median. With the available Kempton relationship, one can perhaps approximate this by focusing on duration between, the median (10.6 seconds) and the 84th percentile (16.5 seconds) values. In this sense the Cerro Prieto and Kakogawa with the duration of 39.6 seconds and 18.2 seconds, respectively, are outliers on the high end, and the Morgan Hill is an outlier on the low end. These outliers are identified by (*) in the above table. The very high duration value of the adjusted Cerro Prieto is again particularly noteworthy.

Figure G2-3 shows the acceleration time histories, response spectral values, and Arias intensity values associated with the Cerro Prieto motion as recorded as well as after adjustments. It is clear from Figure G2-3 that the unusually high Arias intensity value associated with the adjusted Cerro Prieto motion was caused by the as-recorded motion exhibiting unusually long duration combined with relatively low acceleration values generally modified upward to higher spectral values in the adjustment process.

Figure G2-4 shows several recorded acceleration time histories (including the Cerro Prieto motion) in the 1979 Imperial Valley event. The fault broke largely to northwest from the epicenter shown on Figure G2-4. The three acceleration time histories on the left side of the figure that are associated with sites northwest of the epicenter tend to exhibit relatively short duration and relatively high acceleration values. This is in contrast to the three acceleration time histories shown in the right side of the figure that are associated with sites southeast of the epicenter and tend to exhibit relatively long duration and relatively low acceleration levels. The Cerro Prieto motion belongs to the latter group and exhibits the attributes associated with reverse directivity effects (Abrahamson, 2009): relatively low acceleration values, but a very long duration. The results shown on Figure G2-4 provide the reason for the unusually long duration with relatively low acceleration values reflected in the as-recorded acceleration time history shown on Figure G2-3, all of which, when adjusted to a significantly higher response spectrum corresponding to a near-source event, result in the unusually large Arias intensity value shown on Figure G2-3 for the adjusted Cerro Prieto motion.

Figure G2-5 shows the acceleration time histories, response spectral values, and Arias intensity values associated with the Kakogawa motion as recorded as well as after adjustment. It is clear from this figure that the unusually high Arias intensity value associated with the adjusted Kakogawa motion was caused by conditions similar to those for the Cerro Prieto motion; i.e., as-recorded motion exhibiting relatively long duration combined with relatively low acceleration values generally modified upward to higher spectral values in the adjustment process. The results for the Kakogawa motion are not as intense as for the Cerro Prieto motion, but they nevertheless are represent outlier values in terms of Arias intensity and duration.

As can be seen in the above two tables, the Morgan Hill motion is an outlier with Arias intensity and duration values that are too low to be representing the postulated MDE ground motion conditions.

Thus, if one eliminates the Cerro Prieto, Kakogawa, and Morgan Hill motions for the reasons stated above, one is left with the Loma Prieta, Nishi-Akashi, and Northridge motions. As shown in Table G2-3A, these three motions as a group are more than adequately representative of the postulated MDE ground motion conditions particularly with the Loma Prieta motion exhibiting about 98 percentile Arias intensity value.

SUMMARY AND CONCLUSIONS

Based on the data and discussions presented herein, we conclude that the three adjusted acceleration time histories listed in Table G2-3A represent more than adequately the MDE ground motion conditions postulated for the site:

- They all are compatible with the 84th percentile response spectral values associated with the postulated MDE ground motion.
- The values of their earthquake magnitude, distance, etc. are consistent with the MDE ground motion.
- Their Arias intensity values, ranging from 0.7 m/s to 1.4 m/s (or 47 percentile value to 98 percentile value), are consistent with the median and 84th percentile Arias intensity values given the 84th percentile spectral values (a median of 0.7 m/s and an 84th percentile value of 1.0 m/s using the Watson-Lamprey relationship).
- The values of their duration, ranging from 11.0 seconds to 13.6 seconds, are consistent with the MDE ground motion, which has a median duration of 10.6 seconds and an 84th percentile duration of 16.5 seconds using the Kempton relationship.

The first two of the three adjusted time histories listed in Table G2-3B (Cerro Prieto and Kakogawa) were not selected for the following reasons:

- Their Arias intensity values of 2.8 m/s and 1.4 m/s corresponding to 100.0 and 98.5 percentile values are very high values, making these adjusted acceleration time histories truly outliers with respect to the Arias intensity values when compared to the postulated MDE ground motion conditions.

- Their duration being 39.6 and 18.2 seconds are also very high values, making these adjusted acceleration time histories again truly outliers with respect to the duration values when compared to the postulated MDE ground motion conditions.
- Both the Cerro Prieto and the Kakogawa before adjustments exhibit relatively low acceleration values and long durations with higher than 84th percentile Arias intensity value or near median Arias intensity value. When these acceleration time histories are adjusted upward in terms of spectral values, the resulting Arias intensity values become unusually high and the duration remains long. The Cerro Prieto time history is considered to reflect reverse directivity effects, making it relatively low in acceleration values, but very long in terms of its duration.

The adjusted Morgan Hill time history listed in Table G2-3B was not selected because the values of Arias intensity and duration are too low when compared to those of the MDE ground motion.

Finally, it should be noted that the postulated MDE ground motion for the site already reflects considerable amounts of conservatism given the tectonic environment of the site region with the very low slip rate associated with the controlling seismic source. This conservatism includes deterministic 84^{th} percentile response spectral values, earthquake magnitude of 6.5, and a distance of 13.5 km associated with the closest identified fault.

TABLE G2-1

CHARACTERISTICS OF EARTHQUAKE RECORDS SELECTED FOR THE MDE EVENT

No.	Earthquake Event	Recording Station	Style of Faulting (1)	Magnitude (Mw)	Closest Distance (km)	NEHRP Site Classification	Highest Usable Period (sec)	Event Date	Arias Intensity (m/s)	Duration(2) (s)
1	Loma Prieta	San Jose - Santa Teresa Hills	RV/OBL	6.9	14.7	C	15.8	10/18/89	1.3	10.0
2	Northridge	Cataic - Old Ridge Route	RV	6.7	20.7	C	8.3	01/17/94	3.0	8.9
3	Kobe	Nishi - Akashi	SS	6.9	7.1	C	8.0	01/16/95	3.8	9.6
4	Imperial Valley	Cerro Prieto	SS	6.5	15.2	C	10.0	10/15/79	1.3	36.0
5	Morgan Hill	Anderson Dam	DD	6.2	3.3	C	8.0	04/24/84	0.8	6.5
6	Kobe	Kakogawa	SS	6.9	22.5	C	8.0	01/16/95	0.6	17.6

Notes:

(1) SS=Strike-Slip, OBL=Oblique, RV=Reverse or Thrust

(2) Duration based on 5th and 95th percentile time interval using the Arias intensity versus time plot.

(3) MDE ground motion:

Normal fault dipping 75 degrees West

Mw=6.5; Closest distance=13.5 km; Spectral values from deterministic 84th percentile

Arias intensity: Median: 0.7 m/s; 84th Percentile: 1.0 m/s

Significant duration: Median: 10.6 sec.; 84th Percentile: 16.5 sec.

TABLE G2-2

ARIAS INTENSITY, PERCENTILE AND DURATION OF ADJUSTED TIME HISTORIES FOR THE MDE EVENT

No.	Earthquake Event	Recording Station	Arias Intensity		Duration[2] (s)
			Values (m/s)	Percentile[1]	
1	Loma Prieta	San Jose - Santa Teresa Hills	1.4	97.8	11.0
2	Northridge	Cataic - Old Ridge Route	0.7	46.8	13.6
3	Kobe	Nishi - Akashi	0.7	59.9	12.3
4	Imperial Valley	Cerro Prieto	2.8	100.0	39.6
5	Morgan Hill	Anderson Dam	0.4	4.6	6.3
6	Kobe	Kakogawa	1.4	98.5	18.2

Notes:

(1) Percentile determined based on standard error provided in the Watson-Lamprey & Abrahamson Arias intensity relationship

(2) Duration based on 5^{th} and 95^{th} percentile time interval using the Arias intensity versus time plot.

(3) MDE ground motion:

Normal fault dipping 75 degrees West

Mw=6.5; Closest distance=13.5 km; Spectral values from deterministic 84^{th} percentile

Arias intensity: Median: 0.7 m/s; 84^{th} Percentile: 1.0 m/s

Significant duration: Median: 10.6 sec.; 84^{th} Percentile: 16.5 sec.

TABLE G2-3A

ARIAS INTENSITY, PERCENTILE AND DURATION OF TIME HISTORIES SELECTED FOR THE MDE EVENT

No.	Earthquake Event	Recording Station	Arias Intensity		Duration(2) (s)
			Values (m/s)	Percentile(1)	
1	Loma Prieta	San Jose-Sta Teresa Hills	1.4	97.8	11.0
2	Kobe	Nishi - Akashi	0.7	59.9	12.3
3	Northridge	Cataic - Old Ridge Route	0.7	46.8	13.6

TABLE B-3B

ARIAS INTENSITY, PERCENTILE AND DURATION OF TIME HISTORIES NOT SELECTED FOR THE MDE EVENT

No.	Earthquake Event	Recording Station	Arias Intensity		Duration(2) (s)
			Values (m/s)	Percentile(1)	
1	Imperial Valley	Cerro Prieto	2.8	100.0	39.6
2	Kobe	Kakogawa	1.4	98.5	18.2
3	Morgan Hill	Anderson Dam	0.4	4.6	6.3

Notes:

(1) Percentile determined based on standard error provided in the Watson-Lamprey & Abrahamson Arias intensity relationship

(2) Duration based on 5th and 95th percentile time interval using the Arias intensity versus time plot.

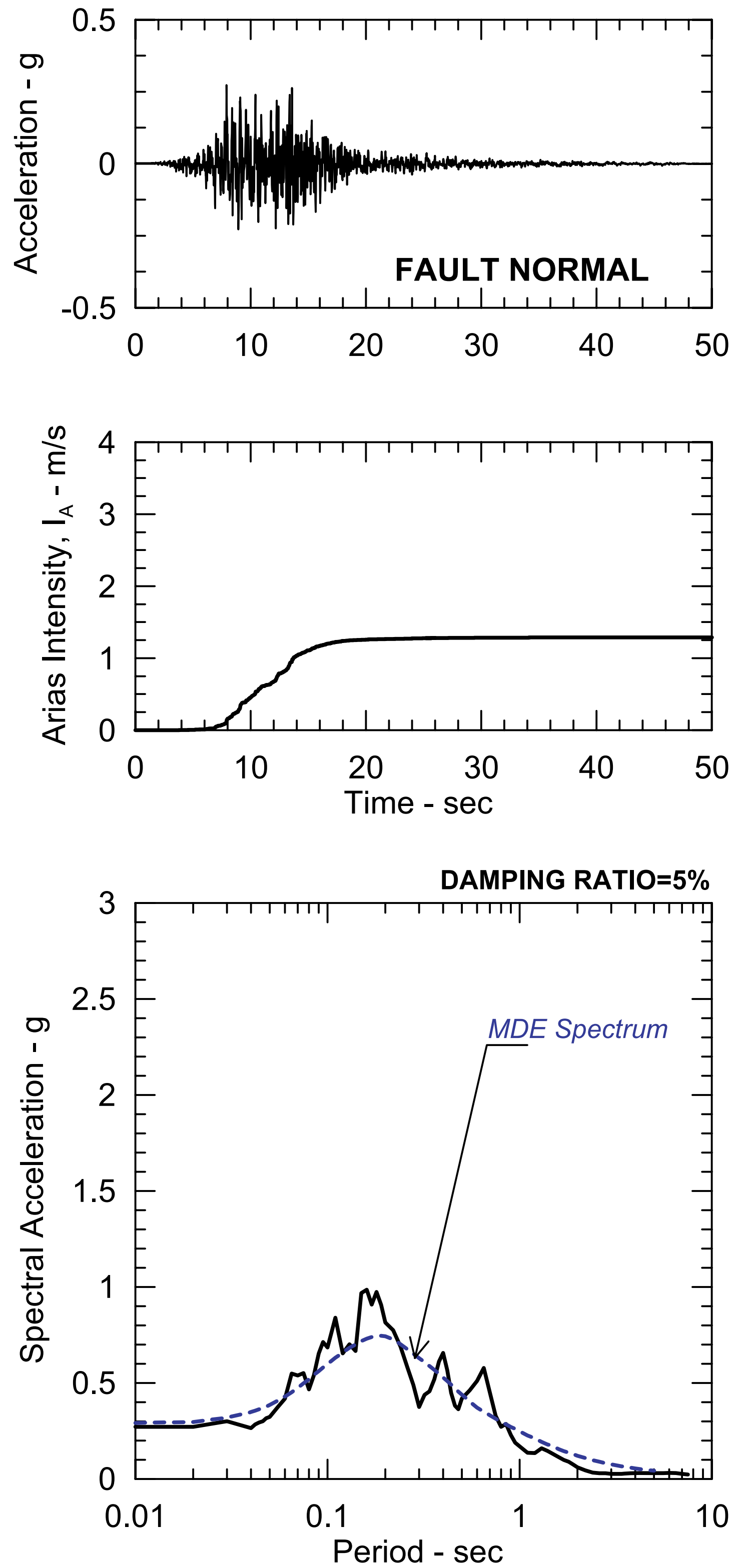
(3) MDE ground motion:

Normal fault dipping 75 degrees West

Mw=6.5; Closest distance=13.5 km; Spectral values from deterministic 84th percentile

Arias intensity: Median: 0.7 m/s; 84th Percentile: 1.0 m/s

Significant duration: Median: 10.6 sec.; 84th Percentile: 16.5 sec.



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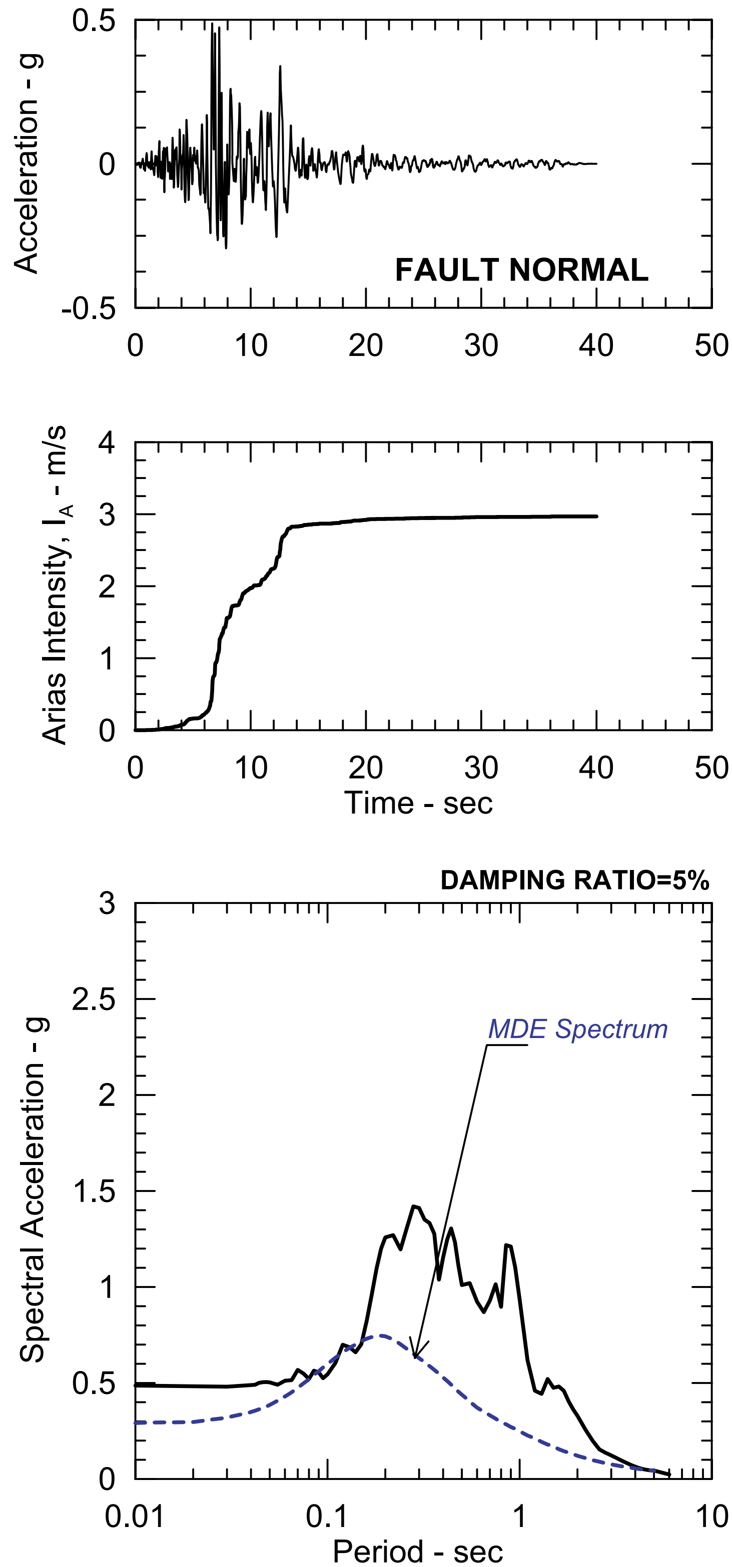
RECORDED TIME HISTORY
AND RESPONSE SPECTRUM
LOMA PRIETA E/Q, SAN JOSE-SANTA THERESA
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-1A



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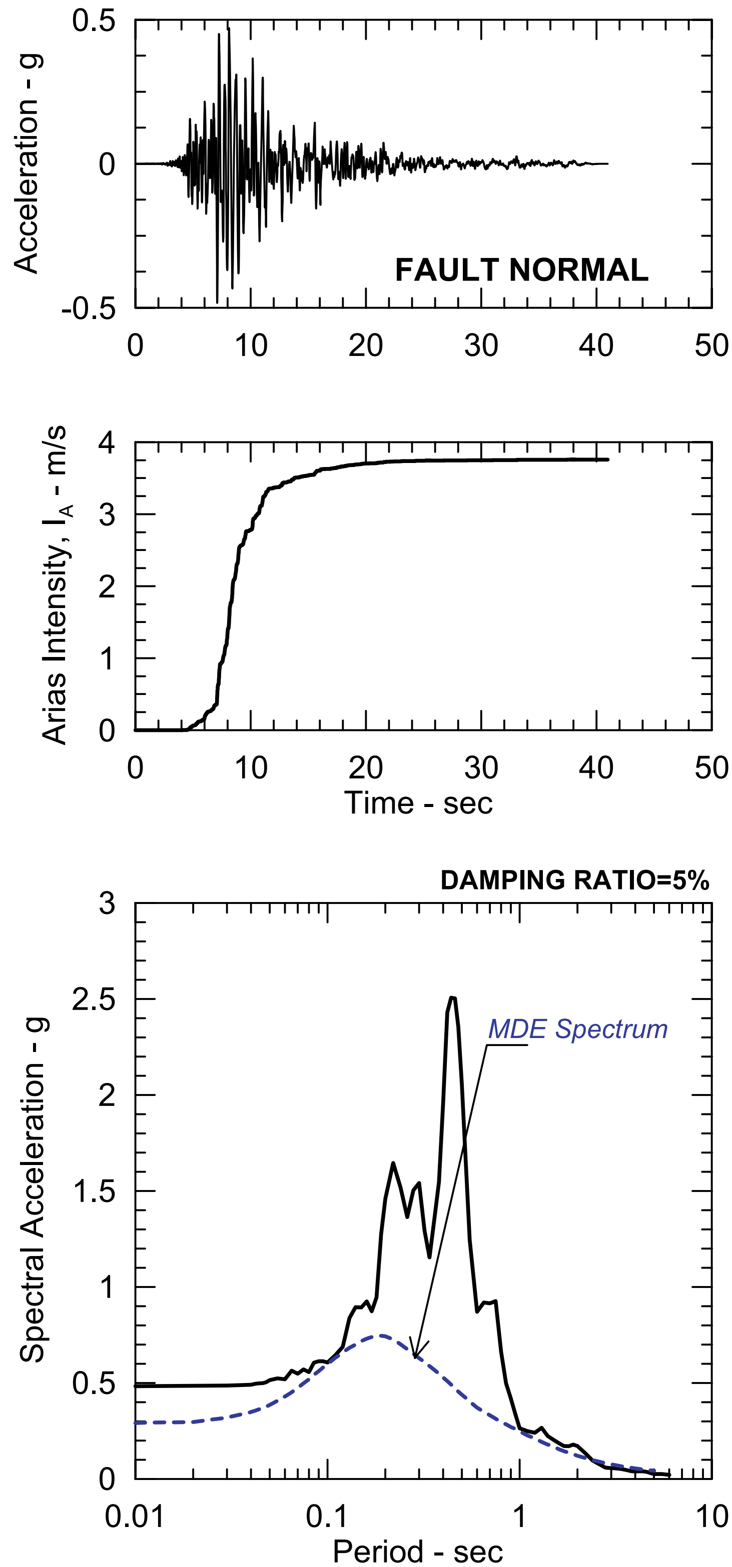
TERRA / GeoPentech a Joint Venture	RECORDED TIME HISTORY AND RESPONSE SPECTRUM NORTHRIDGE E/Q, CASTAIC OLD RIDGE ROAD CAMANCHE EMBANKMENTS SAFETY REVIEW	Figure G2-1B



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RECORDED TIME HISTORY
AND RESPONSE SPECTRUM
KOBE E/Q, NISHI-AKASHI
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-1C

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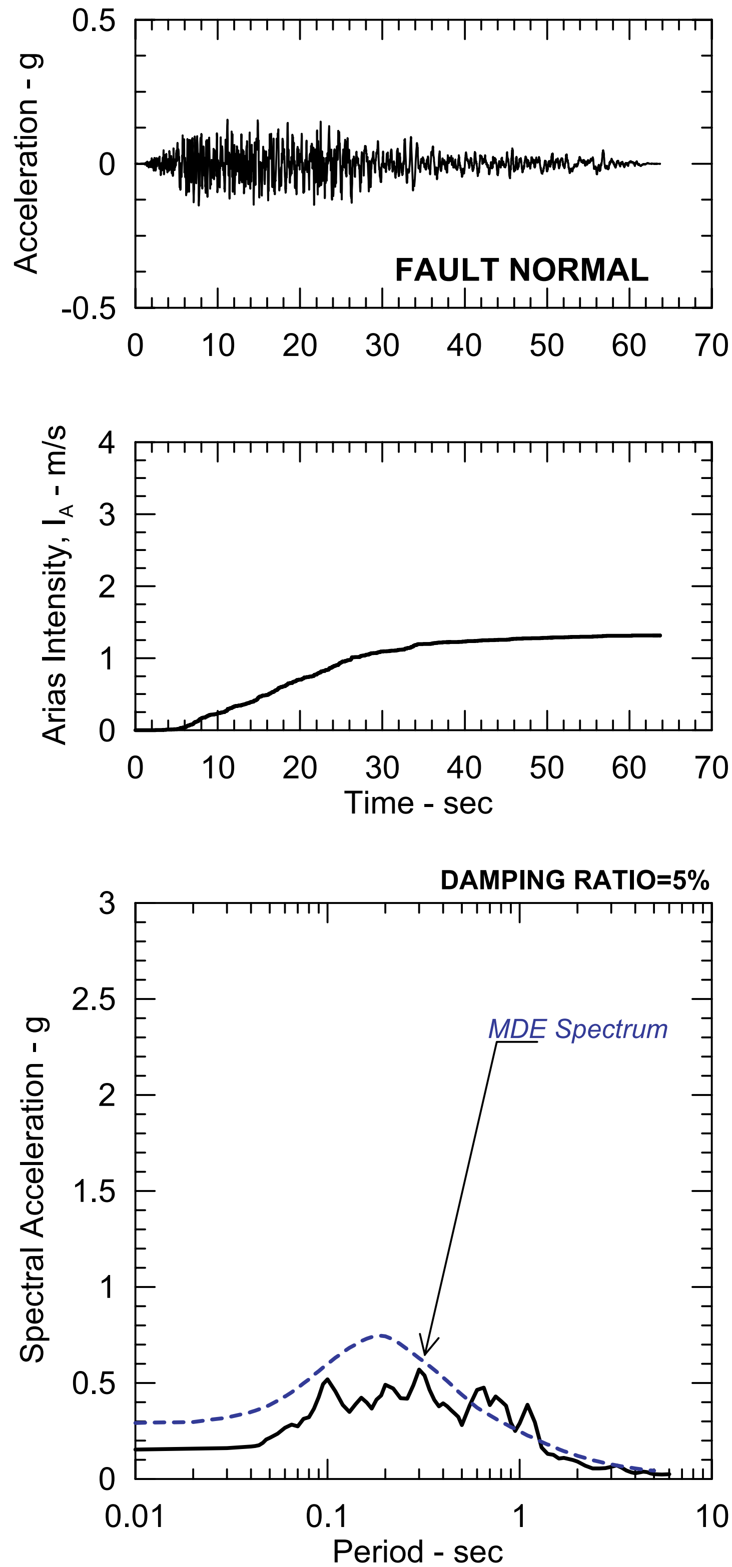

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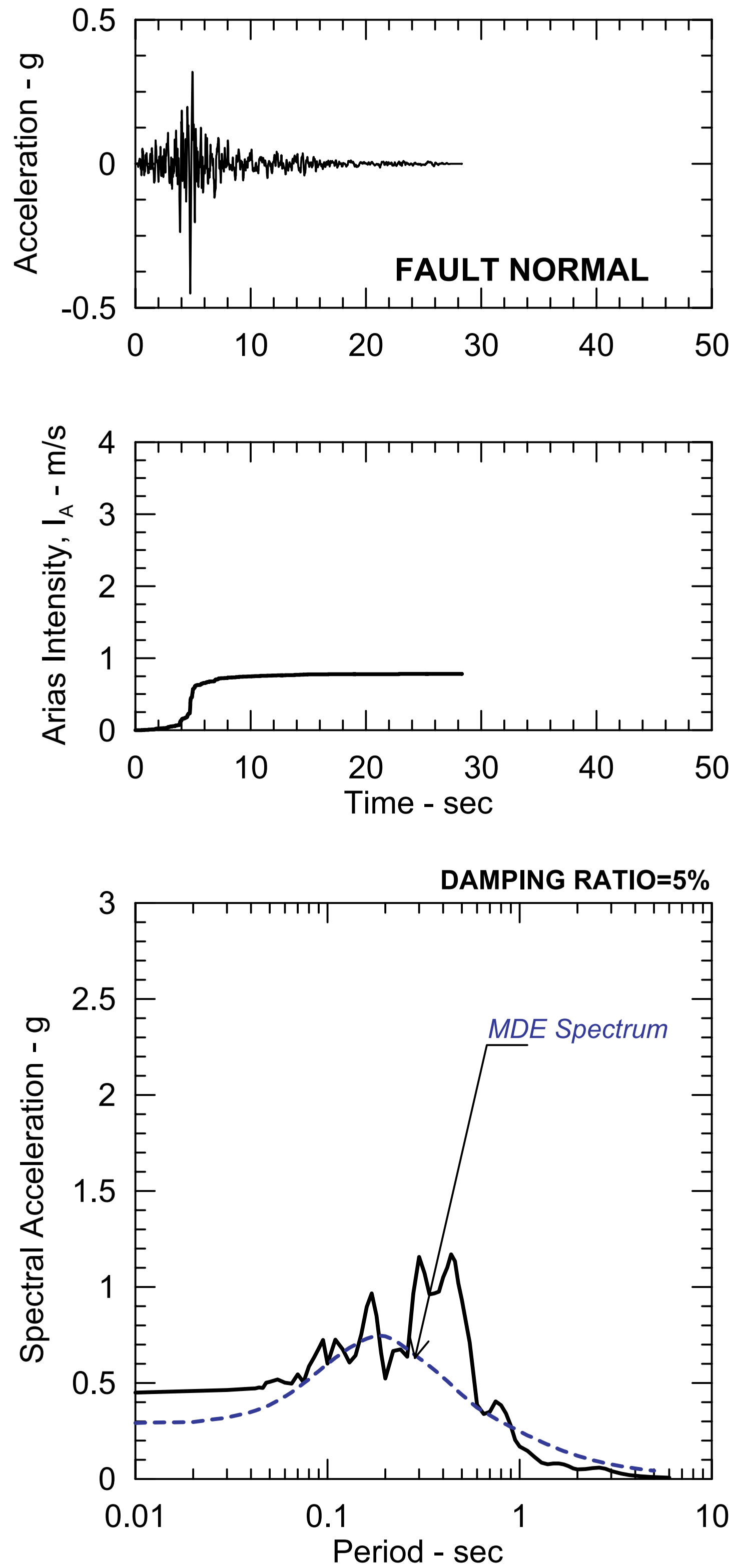
RECORDED TIME HISTORY
AND RESPONSE SPECTRUM
1979 IMPERIAL VALLEY E/Q, CERRO PRIETO
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-1D



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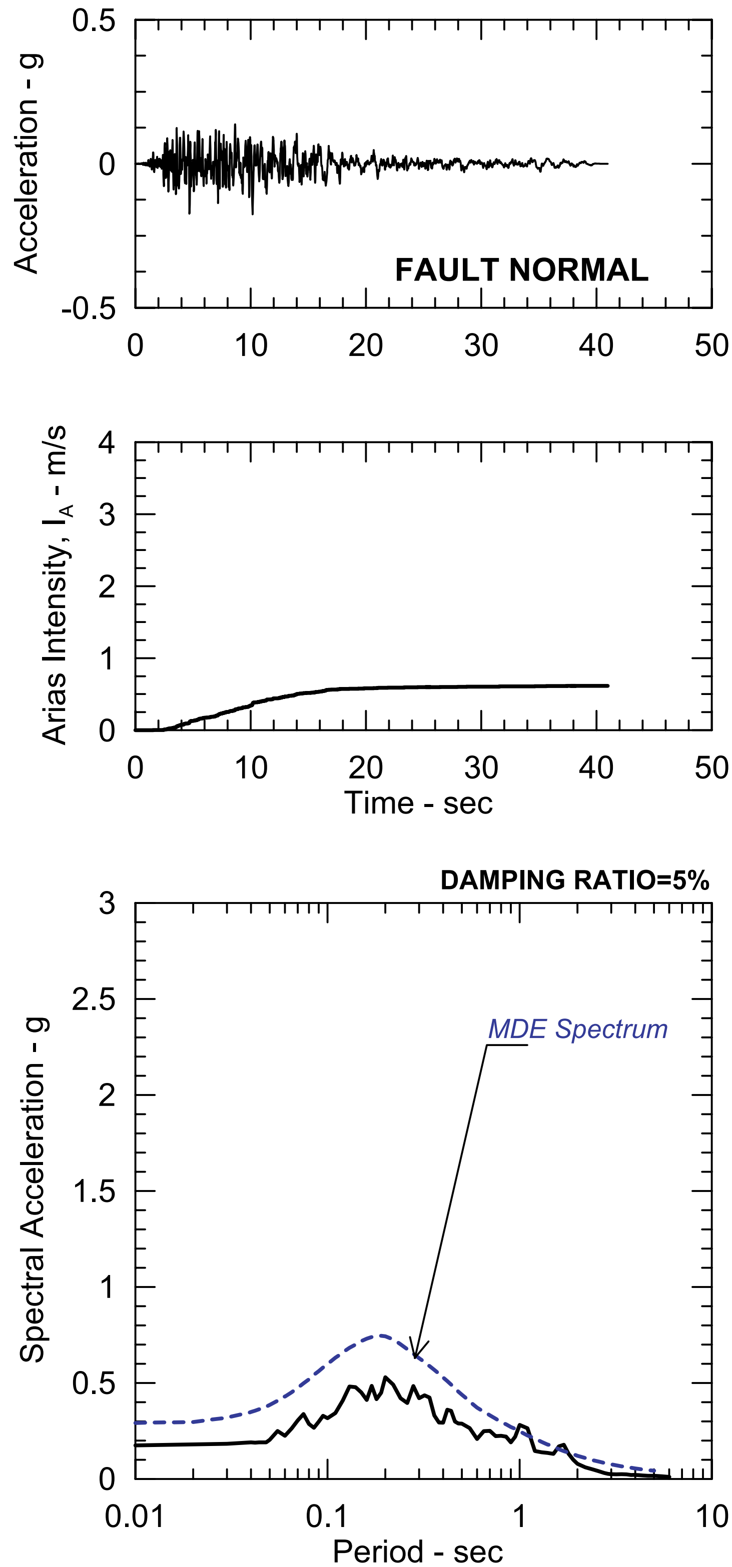
RECORDED TIME HISTORY
AND RESPONSE SPECTRUM
MORGAN HILL E/Q, ANDERSON DAM
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-1E



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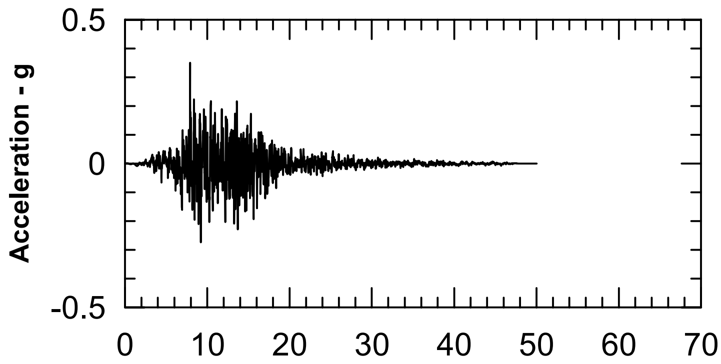
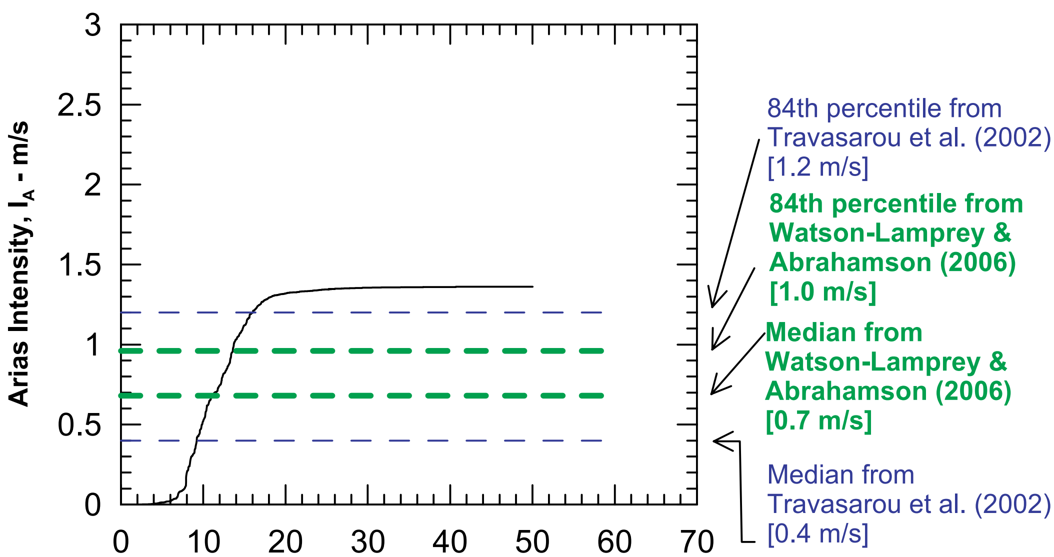
RECORDED TIME HISTORY
AND RESPONSE SPECTRUM
KOBE E/Q, KAKOGAWA
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-1F





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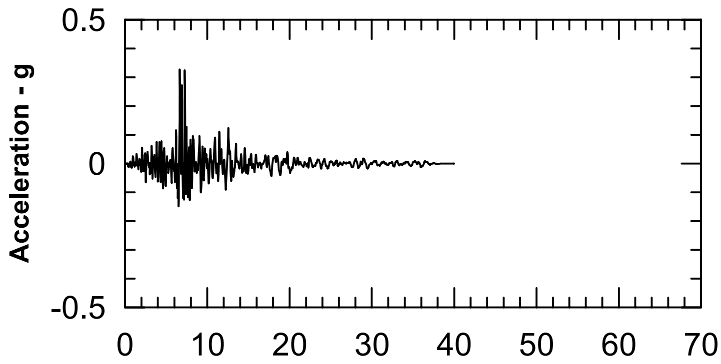
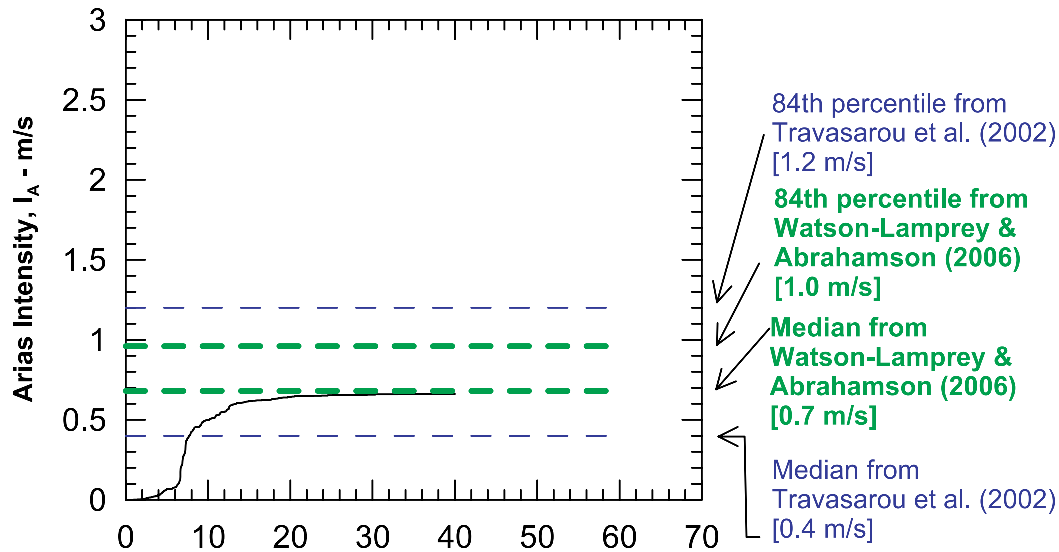
ADJUSTED TIME HISTORY
AND ARIAS INTENSITY
LOMA PRIETA E/Q, SAN JOSE-SANTA THERESA
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-2A





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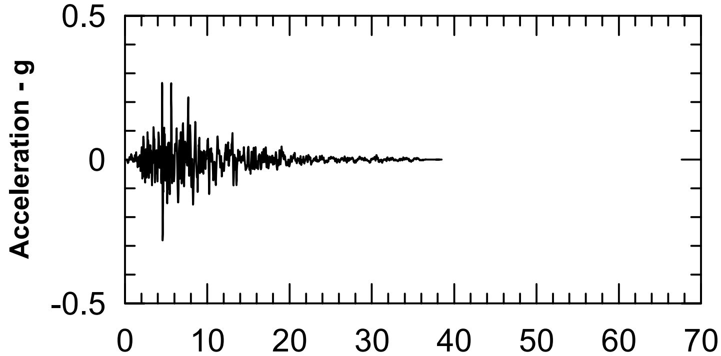
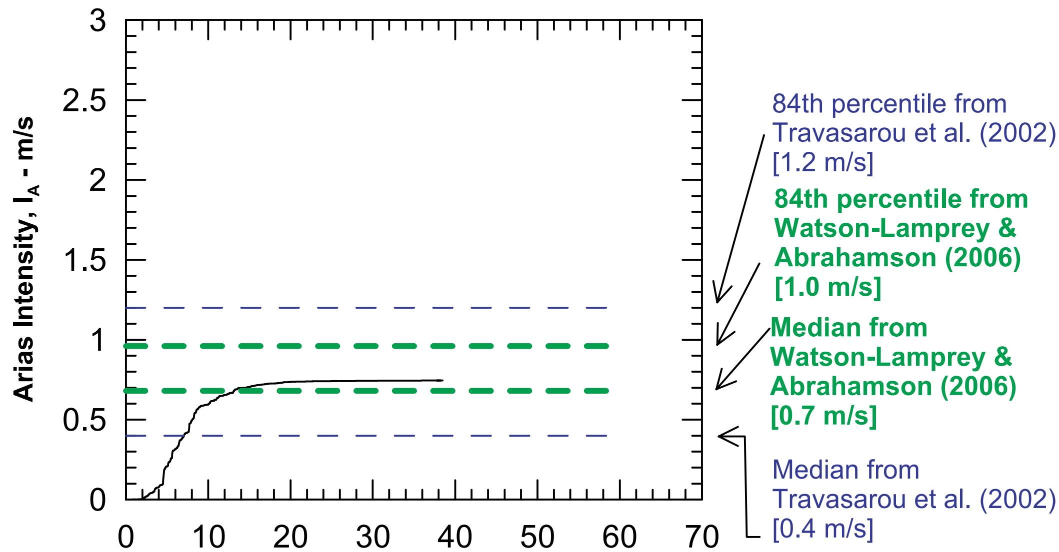
ADJUSTED TIME HISTORY
AND ARIAS INTENSITY
NORTHRIDGE E/Q, CASTAIC OLD RIDGE ROAD
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-2B





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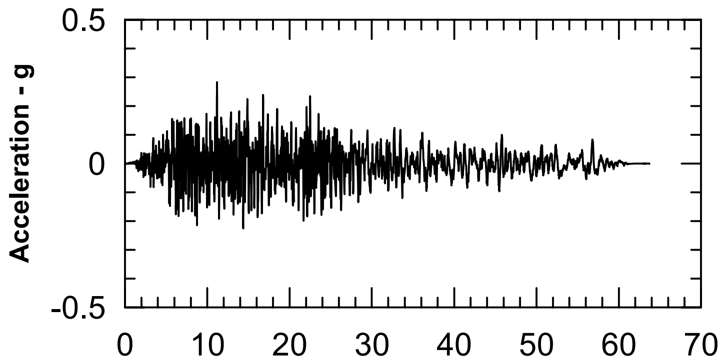
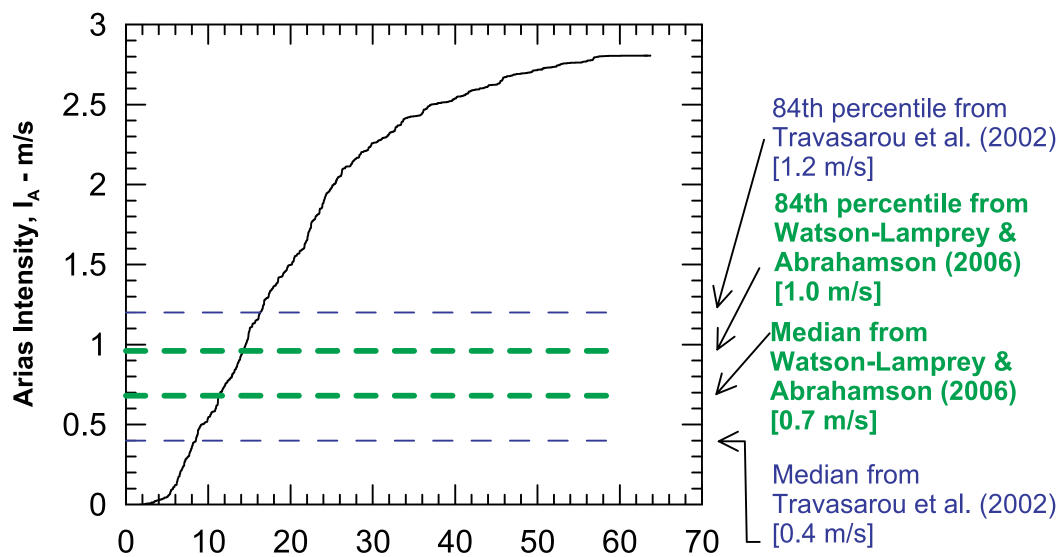
ADJUSTED TIME HISTORY
AND ARIAS INTENSITY
KOBE E/Q, NISHI-AKASHI
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-2C





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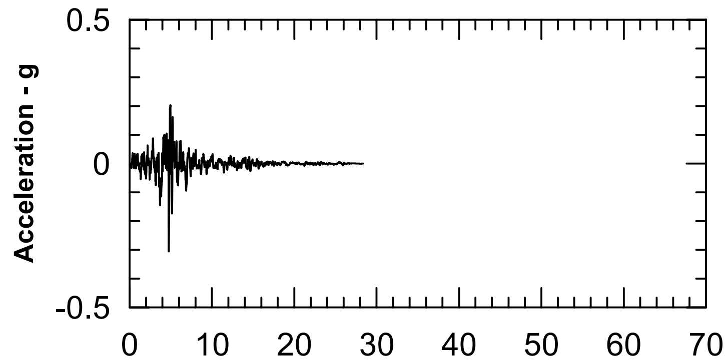
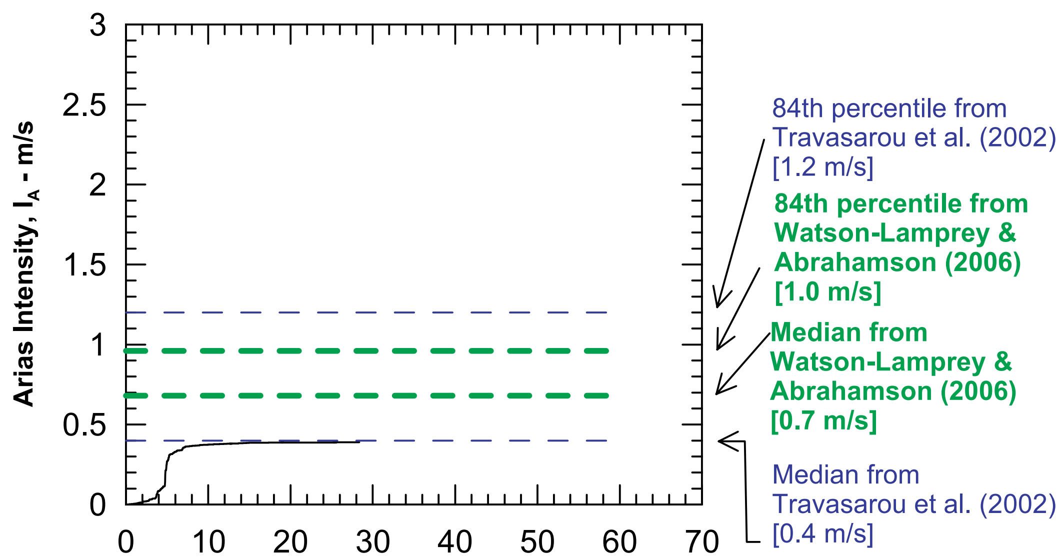
ADJUSTED TIME HISTORY
AND ARIAS INTENSITY
1979 IMPERIAL VALLEY E/Q, CERRO PRIETO
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-2D





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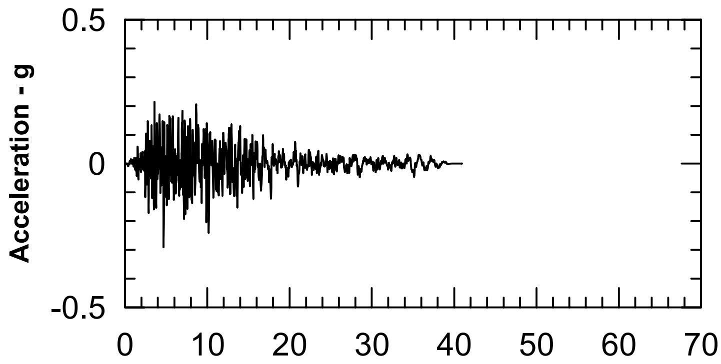
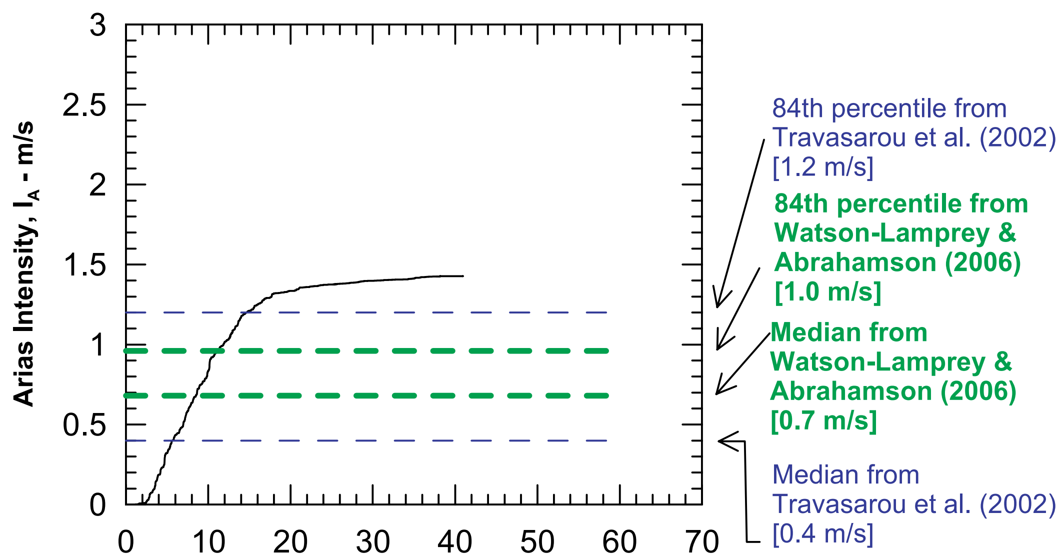
ADJUSTED TIME HISTORY
AND ARIAS INTENSITY
MORGAN HILL E/Q, ANDERSON DAM
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-2E





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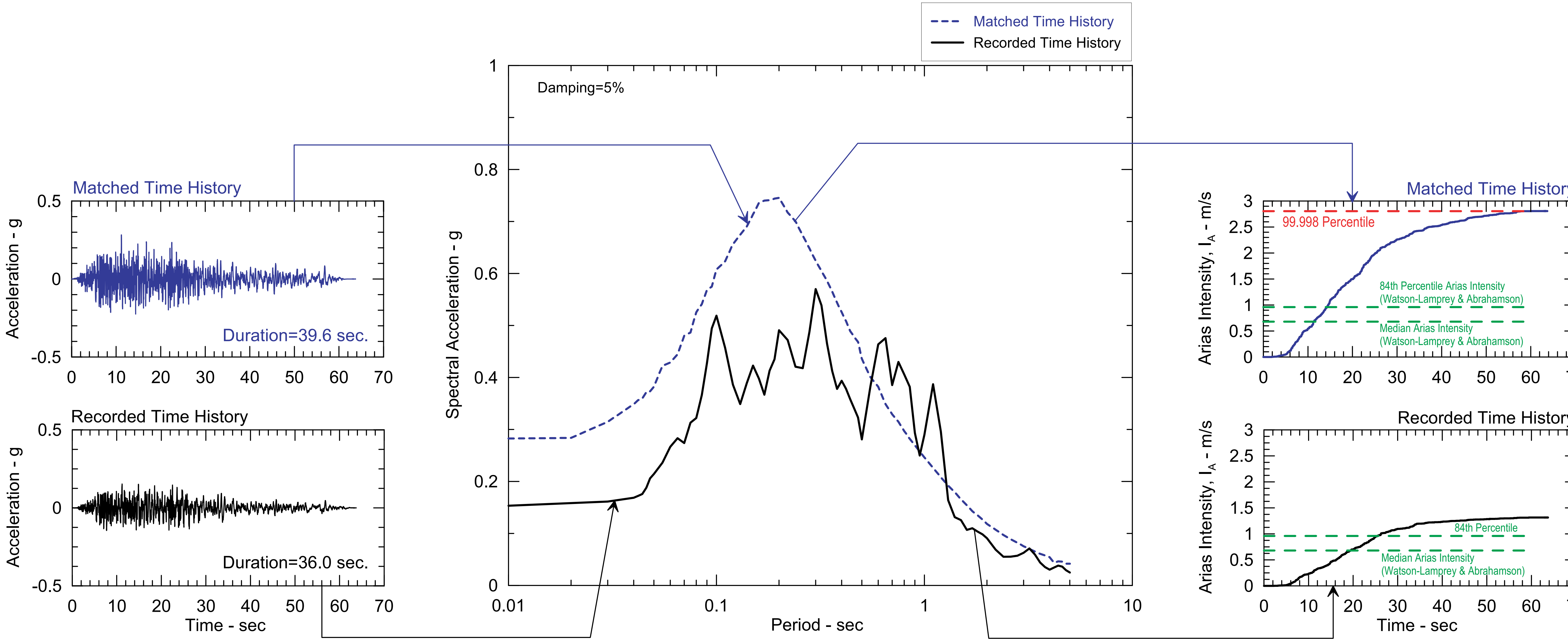
ADJUSTED TIME HISTORY
AND ARIAS INTENSITY
KOBE E/Q, KAKOGAWA
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-2F



Notes:

(1) Duration based on 5th and 95th percentile time interval suing the Arias intensity versus time plot.

(2) MDE ground motion:

Normal fault dipping 75 degrees West

Mw=6.5; Closest distance=13.5 km; Spectral values from deterministic 84th percentile

Arias intensity: Median: 0.7 m/s; 84th Percentile: 1.0 m/s

Significant duration: Median: 10.6 sec.; 84th Percentile: 16.5 sec.

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TIME HISTORY MATCHING
1979 IMPERIAL VALLEY E/Q, CERRO PRIETO
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure G2-3

Rev. 1 02/05/10

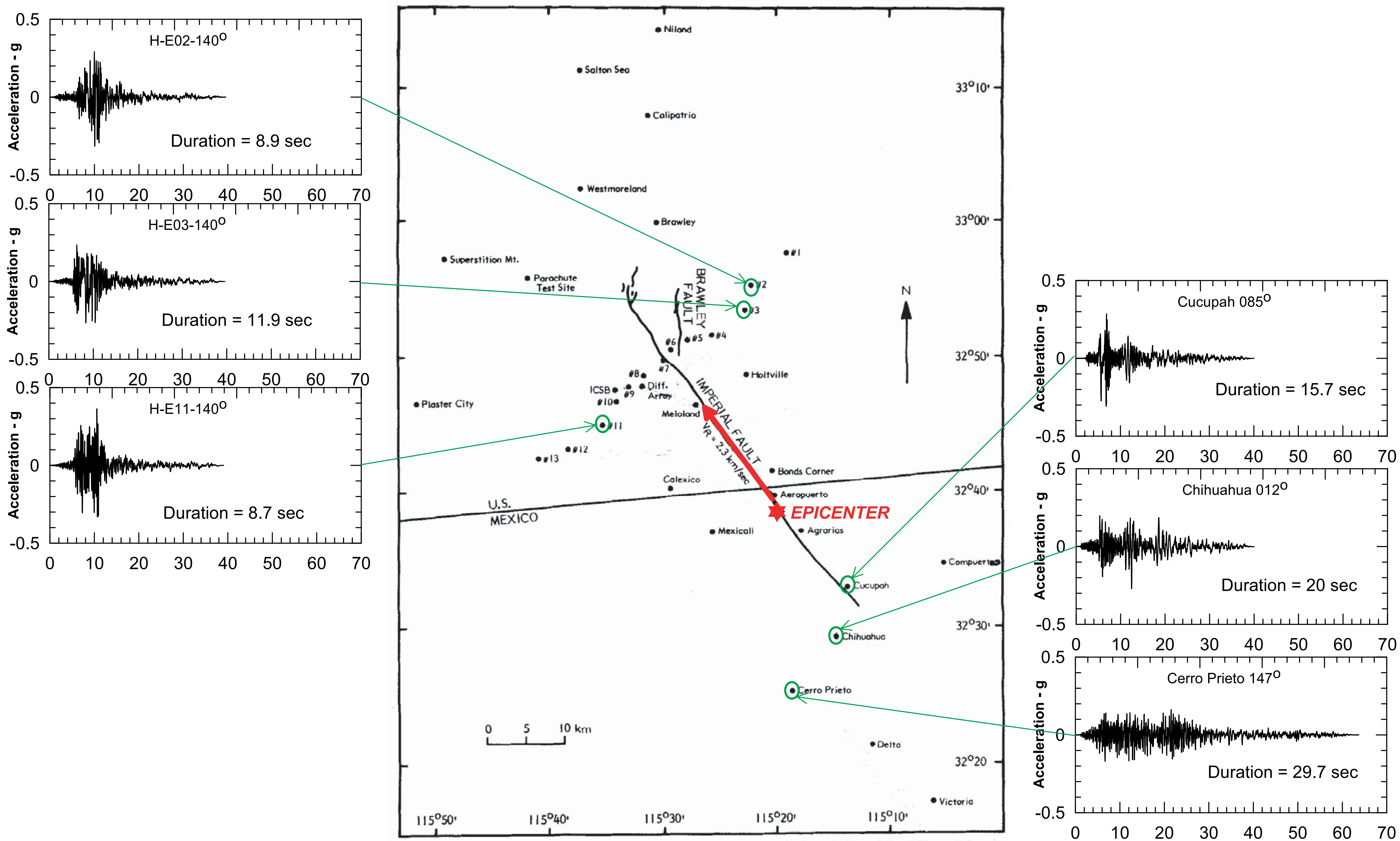
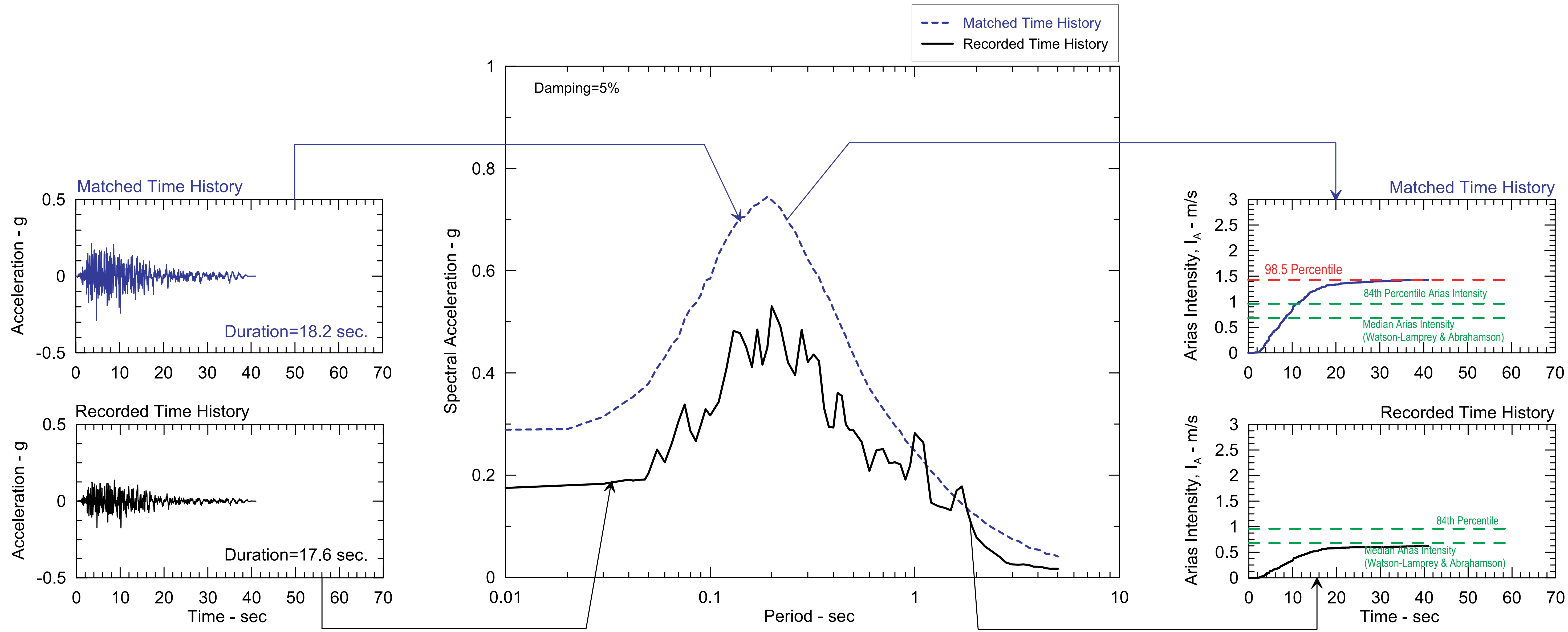


FIG. 7. Location of recording stations used in the empirical analysis and that of the differential array relative to rupture geometry along the Imperial fault.

***Note**: After Niazi M. (1982), BSSA, Vol. 72, No.6, p. 1964*

TERRA / GeoPentech a Joint Venture	RECORDED ACCELERATION TIME HISTORIES AT VARIOUS STATIONS AROUND IMPERIAL VALLEY FAULT CAMANCHE EMBANKMENTS SAFETY REVIEW	Figure G2-4

Rev. 1 02/05/10



Notes:

(1) Duration based on 5th and 95th percentile time interval suing the Arias intensity versus time plot.

(2) MDE ground motion:

Normal fault dipping 75 degrees West

Mw=6.5; Closest distance=13.5 km; Spectral values from deterministic 84th percentile

Arias intensity: Median: 0.7 m/s; 84th Percentile: 1.0 m/s

Significant duration: Median: 10.6 sec.; 84th Percentile: 16.5 sec.

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TIME HISTORY MATCHING
KOBE E/Q, KAKOGAWA
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
G2-5

Rev. 1 02/05/10

ENCLOSURE A
INSTALLATION OF VIBRATING WIRE PIEZOMETERS
AT DIKE 2 – PHASE II

CAMANCHE EMBANKMENTS SAFETY REVIEW

INSTALLATION OF VIBRATING WIRE PIEZOMETERS AT DIKE 2 – PHASE II

REPORT

Prepared for

EAST BAY MUNICIPAL UTILITY DISTRICT
375 Eleventh Street
Oakland, CA 94607

September 2008

TERRA / GeoPentech
a Joint Venture

Black & Veatch
AGS
Natural Resources Consulting Engineers
AP Engineering and Testing

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Appendix

This report describes the construction of six vibrating wire piezometers (VWPs) that were installed in the embankment and foundation of Dike 2 at Camanche Reservoir in June 2008. The work was performed by Terra/GeoPentech, a Joint Venture (Terra/GeoPentech) as part of our Camanche Embankments Safety Review contract with the East Bay Municipal Utility District (District). The installation of these six piezometers completes the Dike 2 piezometer replacement program. This program consists of dual VWP installations in nine borings drilled into the deep alluvial foundation section of Dike 2, with two VWPs installed in each borehole (one each in the embankment and foundation). Phase I of the program (i.e., six of the nine borings) was completed by the District in May and June, 2001. The three remaining borings with six VWPs (i.e., Phase II of the program) were installed in June 2008 as described in this report.

A plan of Dike 2 is presented as Figure 1 which shows the locations of all borings drilled for the VWP installations in 2001 and 2008. Borings D2-32 and D2-33 were drilled from the crest of the dike, 6 feet upstream of the downstream slope hinge point, at Station 252+96 and Station 253+00, respectively. Boring D2-36 was drilled from the top of the downstream berm at Station 252+96. Borings D2-33 and D2-36 were both advanced into the Mehrten bedrock underlying the old alluvium that comprises the dike foundation. Boring D2-32 was terminated after penetrating approximately 25 feet below the top of the old alluvium formation that underlies the embankment.

Figures 2 through 4 summarize lithology and construction details for the 2008 piezometers installed in borings D2-32, D2-33 and D2-36, respectively. The information shown on these figures includes the installed depths of the VWP sensors, the depths of the dike/alluvium contact and the depth of the alluvium/Mehrten contact, and the distribution of boring backfill materials (sand filter sensing zones and bentonite seals). The boring logs are presented in the Appendix which also includes a “Key to Log of Boring”.

Table 1 presents a summary of the VWP locations and installation details for the piezometers installed in 2008. Table 2 presents the VWP serial numbers, check readings taken immediately prior to installation, and the initial readings with calculated head taken after installation.

Descriptions of the drilling and piezometer installation procedures are presented in Section 2.0 of this report.

2.1 DRILLING

Piezometer installation work began on June 9, 2008 and was completed on June 27, 2008. PC Exploration, Inc. of Rocklin, California, under subcontract to Terra/GeoPentech, performed the work including drilling, VWP installation, trench excavation and backfilling for routing the piezometer cables to the terminal boxes, and wiring of the VWP cables at the terminal boxes. The work was performed under full-time supervision and direction of Mr. Richard Harlan, CEG.

Borings D2-32, D2-33 and D2-36 were drilled with an Ingersoll-Rand A400 rotary drill rig. Boring D2-32, which was advanced from the crest of Dike 2 into the upper portion of the alluvial foundation, was drilled to its total depth with 8-inch diameter hollow stem augers. Borings D2-33 and D2-36 were drilled with hollow stem augers to depths of 31 feet and 27 feet below the dike/alluvial foundation contact and were then advanced to completion in the Mehrten Formation via rotary wash drilling using a 3-7/8-inch diameter tri-cone rock bit and Revert biodegradable drill fluid. The hollow stem augers functioned as casing to facilitate piezometer installation and isolate the embankment from rotary drill fluid pressure while advancing the boreholes through the underlying alluvium and Mehrten Formation.

Limited sampling was performed during the course of the drilling. One Standard Penetration Test sample was driven at the bottom of boring D2-32 to provide for examination of the old alluvium comprising the dike foundation at that depth. Additionally, one Modified California sampler was driven at the top of the embankment Zone 1 core in boring D2-33 to obtain several 6-inch long liner samples for laboratory evaluation of core material properties. Intermittent bag samples of mixed auger cuttings were obtained to a depth of 75 feet while drilling through the core on boring D2-32 and tested to determine Atterberg Limits.

Considerable fluid losses initially occurred while advancing the lower portion of boring D2-33 using rotary wash drilling through the old alluvium underlying the embankment. These losses were the result of drilling with a lean Revert-based drill fluid, and some borehole caving occurred prior to piezometer installation after completing the borehole to its total depth of 173 feet. The caved portion of the borehole was re-drilled using a thicker Revert-mix to stabilize the borehole and allow for installation of the VWP tips at their required depths. The lower rotary-drilled portion of boring D2-36 was subsequently drilled with a thick Revert-based fluid mix and the borehole was completed through the alluvium to its total depth in the Mehrten without significant fluid losses or caving.

2.2 DIKE AND FOUNDATION CONDITIONS

Bechtel design drawings 4460-G-40 and 4460-G-41 ("Embankment Explorations – Foundation Dike 2", Sheets 1 and 2) show that Bechtel design boring DH-126 was drilled in the immediate vicinity of D2-32 and D2-33. Those drawings also show the original ground surface at that location was at Elevation 174 feet. This is consistent with the location of the Dike 2 embankment/foundation contact that was encountered at a depth of approximately 93 feet, as interpreted from drill action and first appearance of wet cuttings during the drilling of borings D2-32 and D2-33. The interpreted downstream berm/foundation contact at a depth of 38 feet in boring D2-36 is also consistent with the original ground elevation as shown for this location on Bechtel drawing 4460-G-40.

Borings D2-32 and D2-33 were drilled from the crest of Dike 2 approximately 6 feet upstream from the downstream slope hinge point and encountered gravelly and cobbly material (Zone 6 crest/slope protection) to depths of about 7 feet. The top of the core is at approximately 4 to 5 feet below the crest along Dike 2, and the slightly greater depth of the underlying core as encountered in these borings is attributed to their locations at or just downstream of the hinge point located at the intersection of the top of the core and the downstream slope of the core. The underlying Zone 1 core material was penetrated via stiff and slow drilling with the hollow stem augers. Field and laboratory examination of soil samples and drill cuttings indicate the core material is generally damp to moist, and varies from sandy silts, sandy clays, and silty sands, with low plasticity fines.

The high drill fluid losses encountered while initially drilling through the old alluvium in boring D2-33 using a very thin Revert-based fluid is indicative of a relatively high permeability of the alluvium comprising the Dike 2 foundation. A considerable amount of fine to medium-grained sand, with minimal fines, was present in the fluid return while drilling through the lower alluvial sections in borings D2-33 and D2-36, suggesting the presence of clean sand layers in the deeper alluvium. This is consistent with the earlier Bechtel and Wahler logs that indicated the alluvium is comprised of mostly silty sands and sands. The upper portion of the alluvium was characterized by easier and faster auger penetration in contrast to the stiff, slow drilling through the overlying fine-grained core material in borings D2-32 and D2-33.

The Mehrten Formation was penetrated via slow, steady drilling using the tricone rock bit. The log of Bechtel boring DH-126, which was drilled in close proximity to borings D2-32 and D2-33 indicates that the Mehrten in this local area consists of moderately well-cemented sandstone.

2.3 PIEZOMETER INSTALLATION

The VWP installation generally proceeded according to the following sequence. On completion of the drilling, the rotary wash-drilled borings D2-33 and D2-36 were flushed with a thick Revert-based fluid to stabilize the borehole prior to installation of the VWPs. The VWP cable spools were unwound along the crest of the dike, and both dry and saturated frequency readings were taken for each of the VWPs to check their functioning prior to being lowered into the borehole. VWP check readings taken prior to installation and initial readings with calculated head taken after installation are presented on Table 2. Each VWP was placed in a 2-1/2 inch diameter sand-filled canvas sock and lowered into the borehole by its cable and through the augers to its required depth. The lower VWP in each borehole was placed at depths ranging from 2 to 5 feet above the bottom of the borehole. The VWP was then suspended at that depth in the open borehole while a sand backfill comprised of Lonestar #2/12 was placed via the tremie method up to the top of the desired sensing zone. The lower VWP installed in boring D2-32, which was auger drilled to its total depth without stabilizing drill fluid, was successfully placed at its required depth of 112 feet but was probably partially backfilled with caved native material consisting of saturated sandy alluvium given the relatively small amount of sand that was placed in that interval.

Above the lower sensing zone, the augers were pulled as necessary to allow placement of a bentonite seal up to the bottom of the upper VWP sensing zone. The VWP cable was pulled though each length of auger as the augers were removed. Coated bentonite pellets were slowly dropped down the augers to backfill seal zones below the water table; uncoated pellets were placed for the upper seal zones above water level. The upper VWPs were placed in the same fashion as the lower piezometers and the uppermost portions of the boreholes were typically sealed with bentonite and a cement grout slurry; however the upper seal in boring D2-36 on the downstream berm was backfilled to the surface with bentonite pellets and chips. Frequency readings were taken shortly after installation of each VWP, generally within ½ hour after placing the sand backfill, and again on June 23, 2008, several days after completion of the last installation (see Table 2).

Figures 2 through 4 summarize the Lithology and dual piezometer installation details in borings D2-32, D2-33 and D2-36, respectively.

2.4 PIEZOMETER CABLE TRENCH CONSTRUCTION AND TERMINAL BOX CONNECTION

Following completion of the VWP installations and backfilling of the borings, two trenches (one along the crest, and one along the berm) were excavated to connect the VWPs at Station 253 with the existing terminal boxes in vaults at Station 250. The trenches were excavated by PC Exploration using their JD-310 rubber-tired backhoe with a 12-inch wide bucket. Each of the trenches ranged from 18 to 24 inches in depth, and was typically 12-inches wide along the bottom and flared at the surface to a width of about 24 inches.

The VWP cables were placed within a protective conduit of 2-inch diameter Schedule 80 PVC, and laid along the bottom of the trench. Each trench was then backfilled up to a depth of approximately one foot with a lean, “flowable-fill” type concrete. The upper portion of each trench was then backfilled to the surface with sandy gravel and cobbles that were excavated during trenching, and wheel-rolled with the backhoe. Most of the trench excavation and backfill work was supervised by Mr. Patrick Allen of Terra/GeoPentech.

The VWP cable for the lower piezometer in boring D2-36 (D2-36B) was broken while the driller was pulling the last augers from that borehole, after the installation of both VWPs had been completed. This VWP cable was spliced using a factory splice kit obtained from Durham Geo Slope Indicator prior to installation of the PVC conduit in the lower berm trench. This cable splicing work was performed by Mr. John Stahlecker of PC Exploration.

The VWP piezometer cables were then wired into the terminal boxes along the dike crest and lower berm. Each of the terminal boxes was re-mounted onto the steel lid of its subsurface vault, at the request of the District, to facilitate the periodic readings by District field personnel.

After the cables were wired into the terminal boxes, frequency readings of all of the newly-installed VWPs were again taken on June 27, 2008. As shown on Table 2, these readings were consistent with the previous readings taken on June 23, indicating that piezometric levels in the borehole sensing zones had stabilized.

TABLES

TABLE 1

VIBRATING WIRE PIEZOMETER LOCATIONS AND INSTALLATION DETAILS
DIKE 2 - JUNE 2008

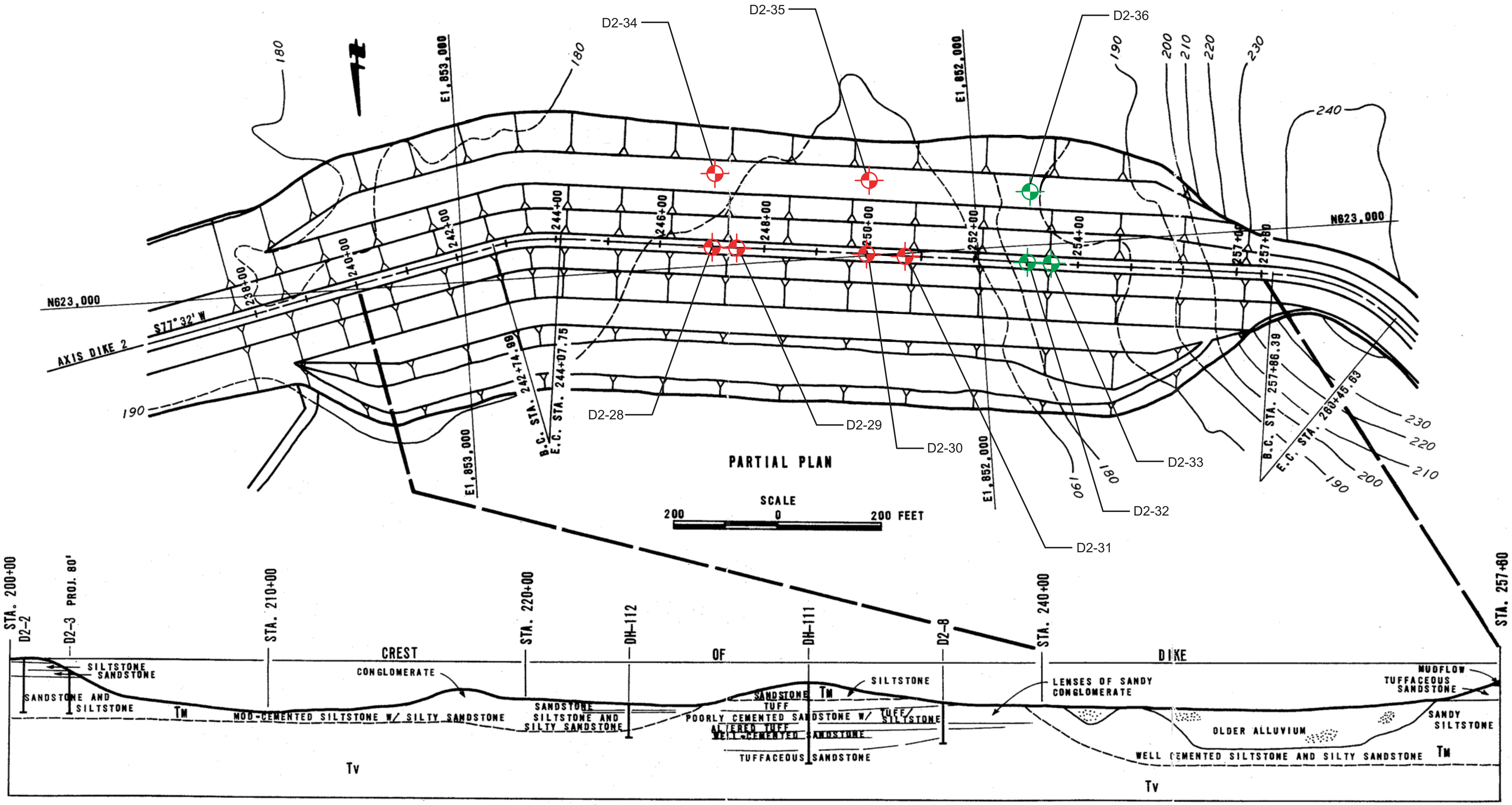
Vibrating Wire Piezometer (Serial Number)	VWP Depth, ft. (Elevation)	Location and Elevation	Sensing Zone Depth, ft	Sensing Zone Location
D2-32A (72332)	81 (El. 185)	Crest at Station 253 El. 266	72-83	Embankment Core
D2-32B (72334)	112 (El. 154)	Crest at Station 253 El. 266	101-116.5	Old Alluvium
D2-33A (72330)	136 (El. 130)	Crest at Sta. 253+20 El. 266	128-140	Old Alluvium
D2-33B (72331)	168 (El. 98)	Crest at Sta. 253+20 El. 266	160-173	Mehrten
D2-36A (72333)	73 (El. 142)	D/S Berm at Sta. 253 El. 215	61-75	Old Alluvium
D2-36B (72335)	108 (El. 107)	D/S Berm at Sta. 253 El. 215	96-110	Mehrten

TABLE 2

VIBRATING WIRE PIEZOMETER INITIAL FREQUENCY READINGS
DIKE 2 – JUNE 2008

Vibrating Wire Piezometer (Serial Number)	Pre-installation Frequency Readings, Hz (VWP at surface)		Initial Installed Frequency Reading (Time, Date)	June 23, 2008 Readings			June 27, 2008 Frequency Readings, Hz
				Measured Frequency, Hz	Calculated		
	Dry	Wet			Pressure Head, psi	Water Depth, ft (Elevation)	
D2-32A (72332)	3041.95	3038.09	3035.81 (5:00 pm, 6/11/08)	3032.93	0	dry	3033.06
D2-32B (72334)	3060.98	3059.15	2983.1 (2:30 pm, 6/11/08)	3019.3	7	96 (El. 170)	3020.17
D2-33A (72330)	3087.19	3085.33	2895.60 (4:35 pm, 6/17/08)	2970.87	17	97 (El. 169)	2971.79
D2-33B (72331)	3076.55	3075.94	2846.66 (4:40 pm, 6/17/08)	2849.51	33	92 (El. 174)	2850.78
D2-36A (72333)	3110.19	3107.68	2951.49 (4:12 pm, 6/19/08)	3033.41	12	45 (El. 170)	3034.13
D2-36B (72335)	3064.16	3061.96	2868.35 (4:10 pm, 6/19/08)	2874.47	27	46 (El. 169)	2875.43

FIGURES



PIEZOMETER BOREHOLES DRILLED IN 2001

PIEZOMETER BOREHOLES DRILLED IN 2008

TERRA / GeoPentech a Joint Venture	PIEZOMETER BORING LOCATION PLAN DIKE 2 CAMANCHE EMBANKMENTS SAFETY REVIEW	Figure 1

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CAMANCHE EMBANKMENTS SAFETY REVIEW	Summary of Lithology and Dual Piezometer Installation in Boring D2-32

Date(s) Drilled	**6/9/08 - 6/11/08**	Date(s) Installed	**6/11/08 - 6/12/08**	Drilled Depth	**116.5 feet**	Surface Elevation	**266 feet MSL**
Water Level Measurements	**Initial reading on 6/11/08 for Tip B at 84 ft, Tip A dry**			Installation Observed By	**R. Harlan**	Installed By	**PC Exploration**
Location	**Dike 2 crest, 6 feet U/S of D/S slope, approx. Station 252+96**			Comments	**See Log of Boring for sample and lab data**		

GROUND SURFACE ELEVATION: 266 feet MSL

CREST / DOWNSTREAM SLOPE PROTECTION (ZONE 6)
COBBLES and WELL-GRADED GRAVEL with SAND (GW), hard, rounded gravel and cobbles to 8 inches (some quartzite); includes pockets/layers of SANDY LEAN CLAY (CL) below 4.5 ft

CORE (ZONE 1)
SANDY LEAN CLAY (CL) and CLAYEY SAND (SC), dark yellowish brown, moist, low plasticity fines, mostly fine-grained sand

... at 35 ft, becomes dark grayish brown

... at 74 ft, several quartzite cobbles (slough?)

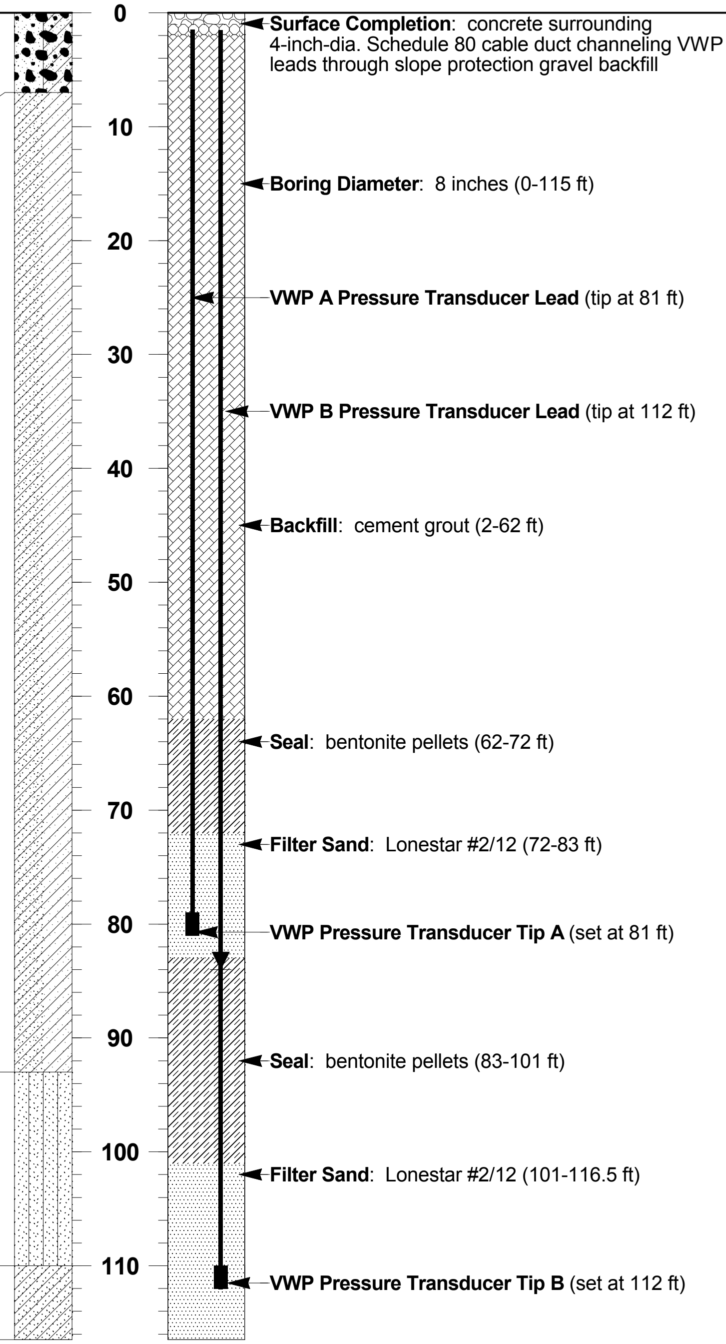
... at 88 ft, becomes very moist

OLD ALLUVIUM / FOUNDATION
SILTY SAND (SM), moderate yellowish brown, wet, mostly fine-grained sand, occasional silt/clay layers

CLAYEY SAND (SC), moderate yellowish brown, wet, fine-grained sand

...at 115 ft, thin layer of SILT (ML), moderate brown

TOTAL DEPTH = 116.5 feet



Report: TGP WELL+LITH; File: TGP_CAMANCHE.GPJ; 8/28/2008

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Figure 2

CAMANCHE EMBANKMENTS SAFETY REVIEW	Summary of Lithology and Dual Piezometer Installation in Boring D2-33

Date(s) Drilled: **6/12/08 - 6/17/08**	Date(s) Installed: **6/17/08 - 6/18/08**	Drilled Depth: **173.0 feet**	Surface Elevation: **266 feet MSL**
Water Level Measurements: **Initial reading on 6/17/08 for Tip B at 92 ft, Tip A at 71 ft**		Installation Observed By: **R. Harlan**	Installed By: **PC Exploration**
Location: **Dike 2 crest, 6 feet U/S of D/S slope, approx. Station 253+20**		Comments: **See Log of Boring for sample and lab data**	

GROUND SURFACE ELEVATION: 266 feet MSL

CREST / DOWNSTREAM SLOPE PROTECTION (ZONE 6)
COBBLES and WELL-GRADED GRAVEL with SAND (GW), hard, rounded gravel and cobbles to 8 inches (some quartzite), occasional clay layers below 5 ft

CORE (ZONE 1)
SANDY LEAN CLAY (CL), CLAYEY SAND (SC), and SANDY SILT (ML), moderate to dark yellowish brown with some orange staining, damp to moist, generally low plasticity fines, mostly fine-grained sand, occasional coarse-grained sand and gravel

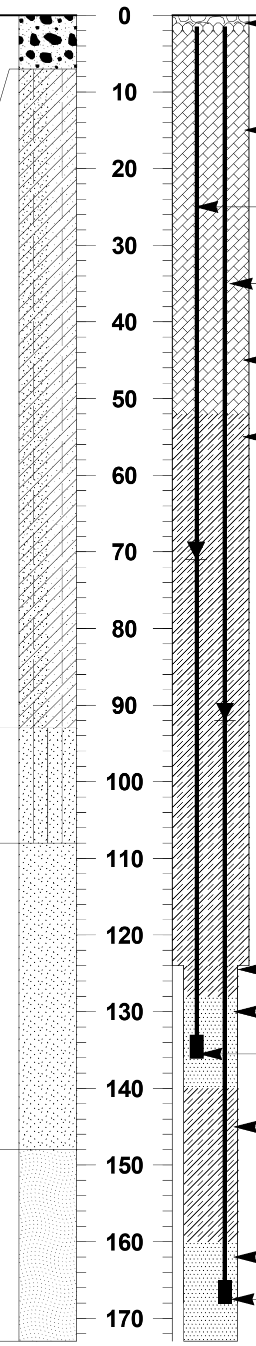
... at 35 ft, becomes dark grayish brown

... at 88 ft, becomes very moist to wet

OLD ALLUVIUM / FOUNDATION
SILTY SAND (SM), moderate yellowish brown, wet; probably includes layers of SANDY SILT (ML), CLAYEY SAND (SC), and POORLY GRADED SAND (SP) (based on earlier Bechtel and Wahler boring logs)

POORLY GRADED SAND (SP), moderate yellowish brown, wet, fine- to medium-grained sand (based on drilling observations and earlier Bechtel and Wahler boring logs)

MEHRTEN FORMATION
SANDSTONE, some andesite (black to gray) rock fragments



TOTAL DEPTH = 173.0 feet

CAMANCHE EMBANKMENTS SAFETY REVIEW	Summary of Lithology and Dual Piezometer Installation in Boring D2-36

Date(s) Drilled	**6/18/08 - 6/19/08**	Date(s) Installed	**6/19/08 and 6/23/08**	Drilled Depth	**110.0 feet**	Surface Elevation	**215 feet MSL**
Water Level Measurements	**Initial reading on 6/19/08 for Tip B at 46 ft, Tip A at 20 ft**			Installation Observed By	**R. Harlan**	Installed By	**PC Exploration**
Location	**Dike 2, 6 feet D/S of D/S slope, inside edge of berm, Sta. 253+00**			Comments	**See Log of Boring for additional data**		

GROUND SURFACE ELEVATION: 215 feet MSL

DOWNSTREAM SLOPE / BERM PROTECTION (ZONE 6)
COBBLES and WELL-GRADED GRAVEL (GW), very hard, rounded, quartzite, quartz, and volcanic cobbles to 10 inches, with fine to coarse gravel and minor sand

RANDOM SHELL (ZONE 4)
WELL-GRADED SAND with GRAVEL (SW), light yellowish brown, dry to damp, fine- to coarse-grained sand, mostly fine gravel, minor fines

CLAYEY SAND (SC), moderate yellowish brown, moist, fine- to medium-grained sand, medium plasticity fines
... at 12 ft, becomes very moist
... at 15 ft, becomes wet

DRAIN AND TRANSITION (ZONES 3 & 2)
POORLY to WELL-GRADED SAND with GRAVEL (SP/SW), wet

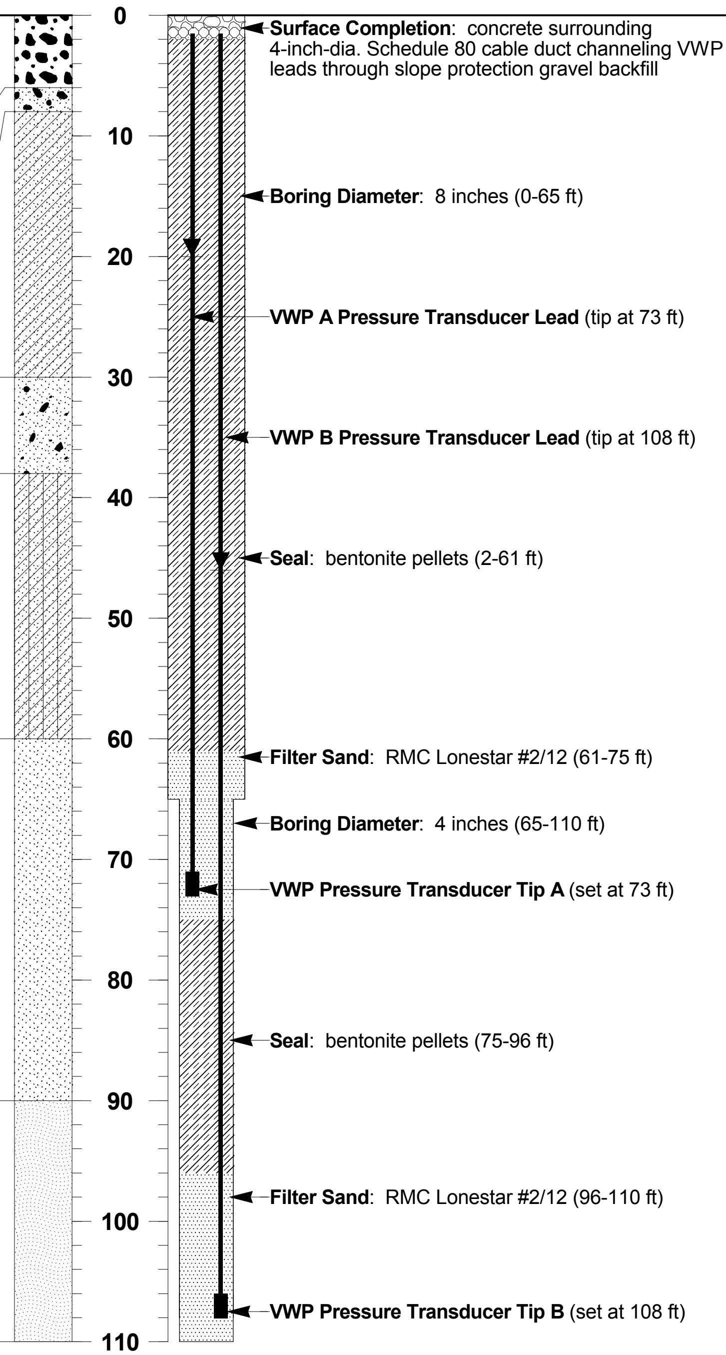
OLD ALLUVIUM / FOUNDATION
SILTY SAND (SM), moderate yellowish brown, wet; probably includes layers of SANDY SILT (ML), CLAYEY SAND (SC), and POORLY GRADED SAND (SP) (based on earlier Bechtel and Wahler boring logs)

POORLY GRADED SAND (SP), light brown, wet, fine- to medium-grained sand; possibly includes occasional gravel layers (based on drilling observations and earlier Bechtel and Wahler boring logs)

... below 80 ft, possibly some gravel layers

MEHRTEN FORMATION
SANDSTONE, some andesitic rock fragments (observed in cuttings)

TOTAL DEPTH = 110.0 feet



Report: TGP WELL+LITH; File: TGP_CAMANCHE.GPJ; 8/28/2008

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Figure 4

APPENDIX

CAMANCHE EMBANKMENTS SAFETY REVIEW	Key to Log of Boring Sheet 1 of 1

Elevation, feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	SAMPLES: Blows Per 6 inches	SAMPLES: Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15

COLUMN DESCRIPTIONS

1 **Elevation:** Elevation in feet referenced to specified datum.

2 **Depth:** Depth in feet below the ground surface.

3 **Sample Type:** Type of soil or rock sample collected at depth interval shown; sampler symbols are explained below.

4 **Sample Number:** Sample identification number.

5 **Blows per 6 inches:** Number of blows required to advance sampler each 6-inch drive interval, or distance noted, using a 140-lb hammer dropped 30 inches.

6 **Recovery:** Length in inches of material recovered in sampler.

7 **Graphic Log:** Graphic depiction of subsurface material encountered. Typical symbols are given below; variations on these symbols are used to indicate secondary soil components.

8 **Material Description:** Description of material encountered; in addition to soil or rock classification, may include color, moisture, relative density/consistency, particle size, and plasticity for soil; texture, weathering, strength, and hardness of bedrock.

9 **% Gravel:** Percent of soil by weight retained on the No. 4 sieve as determined per ASTM Method D422.

10 **% Fines:** Percent of soil by weight passing the No. 200 sieve as determined per ASTM Method D422.

11 **Liquid Limit:** Liquid Limit (LL) of soil specimen passing the No. 40 sieve as determined per ASTM Method D4318.

12 **Plasticity Index:** Plasticity Index (PI=LL-PL) of soil specimen passing the No. 40 sieve as determined per ASTM Method D4318.

13 **Moisture Content:** Moisture content, as a percentage of dry weight of specimen, determined per ASTM Method D2216.

14 **Dry Unit Weight:** Dry weight per unit volume of soil, reported in pounds per cubic foot, determined per ASTM Method D2937.

15 **Field Notes and Other Tests:** Comments and observations regarding drilling or sampling made by driller or field personnel. Lab test results other than those listed in columnar format may be recorded using abbreviations below.

TYPICAL MATERIAL GRAPHIC SYMBOLS

POORLY GRADED SAND (SP)	SILT (ML)	LEAN CLAY (CL)	POORLY GRADED GRAVEL (GP)
WELL-GRADED SAND with SILT (SW-SM)	ELASTIC SILT (MH)	FAT CLAY (CH)	WELL-GRADED GRAVEL (GW)
SILTY SAND (SM)	SANDY SILT (ML)	SANDY LEAN CLAY (CL)	SILTY GRAVEL (GM)
CLAYEY SAND (SC)	SILTY, CLAYEY SAND (SC-SM)	SILTY CLAY (CL-ML)	CLAYEY GRAVEL (GC)

TYPICAL SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT) split spoon
- HQ rock core barrel
- Shelby tube (thin-wall, fixed-head undisturbed)
- Modified California (2.4-inch-ID, 3-inch-OD)
- Pitcher Barrel (lined with Shelby tube)
- Bulk sample collected from auger cuttings

OTHER GRAPHIC SYMBOLS

- First water encountered at time of drilling and sampling
- Static water level measured at specified time after drilling
- Change in material properties within a lithologic unit
- ——— Inferred contact between soil strata or gradational change

OTHER LABORATORY TEST ABBREVIATIONS

CONS One-Dimensional Consolidation Test
CRUMB Crumb Test
TX-UU Unconsolidated Undrained Triaxial Compression Test
TX-CIU Isotropically Consolidated Undrained Triaxial Test
UCS Unconsolidated Compressive Strength Test
%<5μ Hydrometer Test (coupled with Sieve Test)

GENERAL NOTES

1. Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.

2. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

Date(s) Drilled	**6/9/08 - 6/11/08**	Logged By	**R. Harlan**	Checked By	**R. Kirby**
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**8-inch-OD auger**	Total Depth of Borehole	**116.5 feet**
Drill Rig Type	**Ingersoll Rand A400 Rotary**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Wet cuttings below 93 ft ATD; piezometer (Tip B) 84 ft on 6/11/08**	Sampling Method	**Grab from cuttings, SPT (115 ft)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Dike 2 crest, 6 feet U/S of D/S slope, approx. Station 252+96**	Borehole Completion	**Dual vibrating wire piezometers: bentonite 62-72 and 83-101 ft, Tip A at 81 ft in sanded interval 72-83 ft, Tip B at 112 ft in sanded interval 101-116.5 ft**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
	0						**CREST / DOWNSTREAM SLOPE PROTECTION (ZONE 6)** COBBLES and WELL-GRADED GRAVEL with SAND (GW), hard, rounded gravel and cobbles to 8 inches (some quartzite)							Start drilling at 1020 on 6/9/08 with 8-in.-OD hollow-stem auger. Reservoir at El. 196 ft, 70 ft below dike crest. Problem with augers deflecting through cobbles. Move rig 4 ft east, to Sta. 252+96. Destroy three auger bits drilling to 4 ft; end for 6/9/08. Resume drilling at 0730 on 6/10/08. Start with new bit. Smooth drilling below 7 ft.
265														
	5						GRAVELLY SANDY CLAY (CL), yellowish brown Includes pockets/layers of SANDY LEAN CLAY (CL)							
260														
							CORE (ZONE 1) SANDY LEAN CLAY (CL) and CLAYEY SAND (SC), dark yellowish brown, moist, low plasticity fines, mostly fine-grained sand							Retain occasional small plastic bag samples of auger cuttings from 5-ft intervals below 10 ft. Switch auger bit at 11 ft; using bit plug on NW rods.
	10													
255														
			B-1					58	25	12				
	15													
250														
			B-2					64	27	13				
	20													
245														
	25													
240														
			B-3					64	27	14				
	30													

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
	30						SANDY LEAN CLAY (CL) and CLAYEY SAND (SC), dark yellowish brown, moist, low plasticity fines, mostly fine-grained sand [Core - Zone 1] (continued)							
235														
	35						Becomes dark grayish brown							
230														
			B-4					61	25	11				
	40													
225														
	45													Continued smooth, stiff drilling.
220														
	50													
215														
			B-5					65	25	13				
	55													
210														
	60													
205														
			B-6					54	23	10				
	65													

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES
	65					SANDY LEAN CLAY (CL) and CLAYEY SAND (SC), dark grayish brown, moist, low plasticity fines, mostly fine-grained sand [Core - Zone 1] (continued)						Continued smooth, stiff drilling.
200												
	70											Reservoir level at 70 ft below dike crest on 6/9/08.
195												
			B-7				53	24	12			
						Several quartzite cobbles (likely fell in from above)						Rattle at 74 ft; cobbles likely falling in from above.
	75											
190												Slower drilling due to increasing difficulty getting cuttings to surface and cobbles falling in from surface.
	80											End for 6/10/08 at 1645. Resume drilling at 0715 on 6/11/08.
185												
	85											
180												
						Becomes very moist						Driller reports somewhat softer drilling below 88 ft, likely due to increasing moisture content. Wet cuttings first encountered from within 90-95-ft interval.
	90											
175												
						OLD ALLUVIUM / FOUNDATION SILTY SAND (SM), moderate yellowish brown, wet, mostly fine-grained sand, occasional silt/clay layers						
	95											Easier drilling through wet material to hole bottom.
170												
	100											

Elevation, feet	Depth, feet	Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
165							SILTY SAND (SM), moderate yellowish brown, wet, mostly fine-grained sand, occasional silt/clay layers [Old Alluvium / Foundation] (continued)							
160	105													
155	110						CLAYEY SAND (SC), moderate yellowish brown, wet, fine-grained sand							
			B-8					38	26	13				
150	115						SILT (ML), moderate brown							
			SPT-1	67 20 17	18			20	23	10				SPT sample with NWJ rods.
							Bottom of boring at 116.5 feet							Finish drilling and sampling at 1030 on 6/11/08. Start dual piezometer installation; Tip B set at 112 ft at 1410, Tip A at 81 ft at 1700.
							Initial piezometer Tip A (at 81 ft) reading at 1700 on 6/11/08 (just after installation): 3035.81 frequency = 0 psi (dry).							
145	120						Initial piezometer Tip B (at 112 ft) reading at 1430 on 6/11/08 (15 min. after installation): 2983 frequency = 12 psi = 28 ft head (piezometric level at depth of 84 ft).							
140	125													
135	130													
	135													

Date(s) Drilled	**6/12/08 - 6/17/08**	Logged By	**R. Harlan**	Checked By	**R. Kirby**
Drilling Method	**Hollow-Stem Auger 0-124 feet, Rotary Wash 124-173 feet**	Drill Bit Size/Type	**8-inch-OD auger 0-124 feet, 3-7/8-inch tricone 124-173 feet**	Total Depth of Borehole	**173.0 feet**
Drill Rig Type	**Ingersoll Rand A400 Rotary**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Wet cuttings below 8 ft ATD; initial piezometer Tip B 92 ft, Tip A 71 ft**	Sampling Method	**Modified California (2.5-inch-ID), grab from auger cuttings**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Dike 2 crest, 6 feet U/S of D/S slope, approx. Station 253+20**	Borehole Completion	**Dual vibrating wire piezometers: bentonite 52-128 and 140-160 ft, Tip A at 136 ft in sanded interval 128-140 ft, Tip B at 168 ft in sanded interval 160-173 ft**		

Elevation, feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	SAMPLES: Blows Per 6 inches	SAMPLES: Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
	0						**CREST / DOWNSTREAM SLOPE PROTECTION (ZONE 6)** COBBLES and WELL-GRADED GRAVEL with SAND (GW), hard, rounded gravel and cobbles to 8+ inches							Start drilling at 1145 on 6/12/08 using 8-in.-OD hollow-stem auger.
265														
	5						Includes occasional clay layers							
260														
			MC-1A	5	18		**CORE (ZONE 1)** SANDY LEAN CLAY (CL), CLAYEY SAND (SC), and SANDY SILT (ML), moderate to dark yellowish brown with some orange staining, damp to moist, generally low plasticity fines, mostly fine-grained sand, occasional coarse-grained sand and gravel	64	25	11	11.3	99		Smooth drilling at 7 ft. Dark brown clay cuttings. Drive Mod Cal sampler; retain three 6-inch liners. Rocky drilling again at 8 ft; 1/2 hr to grind away large cobble (probably fell in from surface). Resume smooth, stiff drilling below 8 ft.
			MC-1B	16				60	28	13	15.8	106		
			MC-1C	18				36	32	13	16.9	112		
	10													
255														
	15													
250			B-1				[Mixed cuttings appear to be mostly SANDY LEAN CLAY (CL), dark yellowish brown, generally damp to moist, low plasticity fines]							
	20													
245														
	25													
240														
	30													

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
	30						SANDY LEAN CLAY (CL), CLAYEY SAND (SC), and SANDY SILT (ML), moderate to dark yellowish brown with some orange staining, damp to moist, generally low plasticity fines, mostly fine-grained sand, occasional coarse-grained sand and gravel [Core - Zone 1] (continued)							
235														End for 6/12/08 at 1700. Resume drilling at 0720 on 6/13/08. Driller breaks hydraulic line fitting on drilling head; rig down 0730-0915.
	35						Becomes dark grayish brown							
230														
	40													Continued smooth, stiff drilling.
225														
	45													
220														
	50													Driller adds water below 50 ft. Augers start advancing more quickly.
215														
	55													
210														
	60													
205														
	65													

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
200	65						SANDY LEAN CLAY (CL), CLAYEY SAND (SC), and SANDY SILT (ML), dark grayish brown, damp to moist, generally low plasticity fines, mostly fine-grained sand, occasional coarse-grained sand and gravel [Core - Zone 1] (continued)							
195	70													
190	75													
185	80													
180	85						Becomes very moist to wet							Wet cuttings from hole with augers at 88 ft. Continued smooth, stiff drilling.
175	90													Gradually softer drilling below 91 ft.
170	95						**OLD ALLUVIUM / FOUNDATION** SILTY SAND (SM), moderate yellowish brown, wet; probably includes layers of SANDY SILT (ML), CLAYEY SAND (SC), and POORLY GRADED SAND (SP) (based on earlier Bechtel and Wahler boring logs)							Distinctly faster drilling below 93 ft. End for 6/13/08 at 1600. Resume drilling at 0849 on 6/16/08.
	100													

Elevation, feet	Depth, feet	SAMPLES: Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
165							SILTY SAND (SM), moderate yellowish brown, wet; probably includes layers of SANDY SILT (ML), CLAYEY SAND (SC), and POORLY GRADED SAND (SP) [Old Alluvium / Foundation] (continued)							
160	105													
							POORLY GRADED SAND (SP), moderate yellowish brown, wet, fine- to medium-grained sand (based on drilling observations and earlier Bechtel and Wahler boring logs)							Faster drilling below 108 ft, possibly due to decrease in fines. Increased sand in cuttings. Estimated soil classification based on drilling observations and logs of Wahler and Bechtel borings drilled in this area.
155	110													
150	115													Retain bag sample of auger cuttings from 115-118 ft.
145	120													
140	125													Switch from 8-in.-OD augers to rotary wash drilling at 124 ft. Start drilling with 3-7/8-in. tricone bit and biodegradable drilling fluid (Revert) at 1300. Fast drilling to 134 ft. Lost 800 gallons drill fluid from 124-134 ft.
135	130													
	135													Break to get water 1330 to 1445.

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
130							POORLY GRADED SAND (SP), moderate yellowish brown, wet, fine- to medium-grained sand [Old Alluvium / Foundation] (continued)							Approx. 50% drill fluid return below 134 ft. Fast, easy drilling; clean sand cuttings to 148 ft. Rig pump pressure less than 20 psi while rotary drilling.
	140													
125														
	145													
120														
							MEHRTEN FORMATION SANDSTONE, some black to gray andesite rock fragments							Harder, slower drilling below 148 ft. Some hard, black rock fragments (andesite?) in return; mostly grayish brown silt-size cuttings.
	150													
115														
	155													
110														
	160													
105														
	165													Break to get water at 1530, when hole at 164 ft. (Approx. 600 gallons drill fluid lost from 134-164 ft.) Resume drilling at 1628.
100														
	170													

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Elevation, feet	Depth, feet	Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
95							SANDSTONE, some black to gray andesite rock fragments [Mehrten Formation] (continued)							Finish hole to total depth at 171 ft on 6/16/08 at 1645.
90	175						**Bottom of boring at 173.0 feet** Initial piezometer Tip A (at 136 ft) reading at 1635 on 6/17/08 (45 min. after installation): 2896 frequency = 28 psi = 65 ft head (piezometric level at depth of 71 ft). Initial piezometer Tip B (at 168 ft) reading at 1640 on 6/17/08 (2 hrs 10 min. after installation): 2847 frequency = 33 psi = 76 ft head (piezometric level at depth of 92 ft).							Return for piezometer installation on 6/17/08, but hole had caved up to 130-ft depth overnight. Complete redrilling hole to 173 ft at 1300 on 6/17/08. Start dual piezometer installation; Tip B set at 168 ft, Tip A set at 136 ft. Finish grouting to surface at 1230 on 6/18/08.
85	180													
80	185													
75	190													
70	195													
65	200													
	205													

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Date(s) Drilled	6/18/08 - 6/19/08	Logged By	R. Harlan	Checked By	R. Kirby
Drilling Method	Hollow-Stem Auger 0-65 feet, Rotary Wash 65-110 feet	Drill Bit Size/Type	8-inch-OD auger 0-65 feet, 3-7/8-inch tricone 65-110 feet	Total Depth of Borehole	110.0 feet
Drill Rig Type	Ingersoll Rand A400 Rotary	Drilling Contractor	PC Exploration	Approximate Surface Elevation	215 feet MSL
Groundwater Level(s)	Wet cuttings below 15 ft ATD; initial piezometer Tip B 46 ft, Tip A 20 ft	Sampling Method	No samples collected	Hammer Data	Not applicable
Borehole Location	Dike 2, 6 feet D/S of D/S slope, inside edge of berm, Station 253+00	Borehole Completion	Dual vibrating wire piezometers: bentonite 0-61 and 75-96 ft, Tip A at 73 ft in sanded interval 61-75 ft, Tip B at 108 ft in sanded interval 96-110 ft		

Elevation, feet	Depth, feet	SAMPLES: Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
215	0						**DOWNSTREAM SLOPE / BERM PROTECTION (ZONE 6)** COBBLES and WELL-GRADED GRAVEL (GW), very hard, rounded, quartzite, quartz, and volcanic cobbles to 10 inches, with fine to coarse gravel and minor sand							Start drilling at 1550 on 6/18/08 with 8-in.-OD hollow-stem auger.
210	5													
							RANDOM SHELL (ZONE 4) WELL-GRADED SAND with GRAVEL (SW), light yellowish brown, dry to damp, fine- to coarse-grained sand, mostly fine gravel, minor fines							Easier drilling, increased sand cuttings below 6 ft.
							CLAYEY SAND (SC), moderate yellowish brown, moist, fine- to medium-grained sand, medium plasticity fines							
205	10													
							Becomes very moist							
200	15						Becomes wet							Wet cuttings below 15 ft. End for 6/18/08. Resume drilling at 0720 on 6/19/08. Reservoir level at El. 195.2 ft at start of drilling.
195	20													
190	25													Stiff augering through clayey sand shell.
185	30													

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
185	30						**DRAIN AND TRANSITION (ZONES 3 & 2)** POORLY to WELL-GRADED SAND with GRAVEL (SP/SW), wet							Fluid cuttings at 30 ft. Continued stiff drilling. Hole caves at 32 ft; auger back through. Occasional auger chatter (due to gravel) from 30-38 ft.
180	35													
							OLD ALLUVIUM / FOUNDATION SILTY SAND (SM) and CLAYEY SAND (SC), brown, wet, fine- to medium-grained sand; probably includes layers of POORLY GRADED SAND (SP) (based on auger cuttings and earlier Bechtel and Wahler boring logs)							Faster drilling at 38 ft.
175	40													
170	45													Continued wet, clayey sand cuttings from hole.
165	50													
160	55													
155	60						POORLY GRADED SAND (SP), light brown, wet, fine- to medium-grained sand; possibly includes occasional gravel layers (based on drilling observations and earlier Bechtel and Wahler boring logs)							
150	65													

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
150	65						POORLY GRADED SAND (SP), light brown, wet, fine- to medium-grained sand; possibly includes occasional gravel layers [Old Alluvium / Foundation] (continued)							Switch from 8-in.-OD augers to rotary wash drilling at 65 ft. Start drilling with 3-7/8-in. tricone bit and biodegradable drilling fluid (Revert) at 1200.
145	70													
140	75													
135	80						Possibly includes some gravel layers							Some zones of harder drilling below 80 ft.
130	85													Losing only minor fluid; drilling with much thicker Revert mix than at D2-33.
125	90						**MEHRTEN FORMATION** SANDSTONE, some andesitic rock fragments (observed in cuttings)							Distinctly harder, slower drilling below 90 ft.
120	95													
115	100													

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES
							SANDSTONE, some andesitic rock fragments [Mehrten Formation] (continued)							
110	105													
105	110						**Bottom of boring at 110.0 feet** Initial piezometer Tip A (at 73 ft) reading at 1612 on 6/19/08 (45 min. after installation): 2951 frequency = 23 psi = 53 ft head (piezometric level at depth of 20 ft). Initial piezometer Tip B (at 108 ft) reading at 1610 on 6/19/08 (1 hr 40 min. after installation): 2868 frequency = 27 psi = 62 ft head (piezometric level at depth of 46 ft).							Finish drilling and sampling at 1342 on 6/19/08. Start dual piezometer installation; Tip B set at 108 ft at 1430, Tip A at 73 ft at 1520. Finish upper bentonite seal on 6/23/08.
100	115													
95	120													
90	125													
85	130													
80	135													

ENCLOSURE B
PHASE I FIELD INVESTIGATION AND LABORATORY TESTING

EVALUATION OF POTENTIAL FOR DESICCATION CRACKING AND INTERNAL EROSION OF CORE MATERIALS

CAMANCHE EMBANKMENTS SAFETY REVIEW

PHASE I FIELD INVESTIGATION AND LABORATORY TESTING

EVALUATION OF POTENTIAL FOR DESICCATION CRACKING AND INTERNAL EROSION OF CORE MATERIALS

REPORT

Prepared for

EAST BAY MUNICIPAL UTILITY DISTRICT
375 Eleventh Street
Oakland, CA 94607

January 2009

TERRA / GeoPentech
a Joint Venture

Black & Veatch
AGS
Natural Resources Consulting Engineers
AP Engineering and Testing

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Appendices

1.1 INTRODUCTION

This report describes field and laboratory investigations conducted to evaluate the potential for desiccation cracking and internal erosion of the upper "Zone 1" core materials within the Main Dam and dike embankments at Camanche Reservoir. This investigation was performed by Terra/GeoPentech as part of our Camanche Embankments Safety Review contract with East Bay Municipal Utility District (EBMUD).

The Camanche Embankments Safety Review includes evaluation of piping potential under normal and extreme reservoir levels. One piping mechanism that was considered potentially feasible is the concentrated flow of water, during the PMF, through transverse cracks in the dam that may have formed due to desiccation of the upper portions of the clay core. Prior to commencing this study, our initial review of available data indicated that the presence or absence of desiccation cracks in the upper portion of the core was unknown, and data on the susceptibility to internal erosion of the clay core was also lacking. This investigation was therefore designed to evaluate the potential for this mechanism to develop by determining the actual presence or absence of cracking in the upper core and evaluating the susceptibility of the core material to internal erosion.

The field investigation consisted of drilling 11 borings and excavation of three test pits from approximately the center of the embankment crests. Two borings each were drilled on the Main Dam and Dikes 1, 2, and 3, and one boring each was drilled on Dikes 4, 5 and 6. As shown on Figure 1, the borings were located at approximately the one-third and two-third station points along each of the longer embankments that have two borings each (Dikes 1, 2, and 3, and the Main Dam), and at the center of the shorter embankments that have one boring each (Dikes 4, 5, and 6). The borings were advanced via hollow stem auger drilling with drive sampling to provide blow count data and samples of the upper core, and were drilled to total depths of approximately 15 feet (10 feet into the core beneath the 4 to 5-foot thick layer of "Zone 6" gravelly slope protection on the crest of the embankments). The three test pits were excavated to allow visual examination of the core materials, primarily to determine the presence or absence of desiccation cracking. Each of the test pits was located adjacent to one of the borings drilled on the Main Dam and at Dikes 2 and 5. The test pit locations are also shown on Figure 1.

The laboratory investigation included testing of the boring samples for measurement of water content, Atterberg Limits, grain size analyses, dispersivity tests, and pinhole erosion tests.

The field phase of this investigation commenced on July 16, 2008 and was completed on July 23, 2008. Review and approval of the investigation plan was provided by the District and DSOD prior to the start of work. The boring samples were submitted for testing to AP Engineering and Testing, Inc. on July 24, 2008.

Descriptions of the drilling and test pit excavation are presented in the following Section 2, which includes a brief description of the upper embankment conditions as encountered in the borings and test pits. The boring and test pit logs are presented in Appendix A, and locations of the borings and test pits are depicted on Figure 1. Selected photos of the upper shell and core zone materials as exposed in the test pits, along with photos of the excavation and backfilling, are shown as Figures 2 through 12. Laboratory test data are presented in Appendix B and discussed in Section 3; Table B-1 of Appendix B presents a summary of the laboratory data.

1.2 SUMMARY

As discussed in Sections 2 and 3 of this report, we have concluded that:

- No indications of desiccation cracking or other open cracks or seams were noted within the core materials exposed along the walls of the test pits.
- The moisture content data from the laboratory testing program show there is no apparent trend of increasing moisture content with depth within the upper 10 ft of the core material and therefore no indication that drying of the upper portion of the core due to desiccation has occurred.
- The Crumb Tests indicated that some of the soil samples have the potential for being dispersive but the Pinhole Dispersion Tests show that the soils are not erodible when subjected to concentrated flow of water. Consequently, the soils are classified as non-dispersive for the purpose of our evaluation of piping potential.

2.1 DRILLING

The drilling phase of the 2008 Phase 1 field investigation commenced on July 16 and was completed on July 18, 2008. PC Exploration, Inc. of Rocklin, California, under subcontract to Terra/GeoPentech, performed the drilling as well as the subsequent test pit excavation. The work was performed under full-time supervision and direction of Mr. Richard D. Harlan, RG, CEG.

Borings DH-201 through DH-211 were drilled with an Ingersoll-Rand A400 rotary drill rig. Most of the holes were drilled and sampled to a total depth of 14.5 feet; hole depths ranged from 14 to 16.5 feet. Six-inch diameter hollow stem augers were used to advance the borings through the 4- to 5-foot thick gravelly and cobbly “Zone 6” slope protection, and underlying “Zone 1” core. Each boring was advanced approximately 10 feet into the core.

The core was sampled by driving a 2.4-inch inside diameter (3.0-inch outside diameter) California sampler for a distance of 18 inches, using an automatic 140-lb. hammer with 30-inch drop. Blow counts were recorded for each of the typically three 6-inch intervals driven. Each sample was driven at 2-foot intervals so that 5 sampling drives were performed within the 10-ft. length of core sampled in each hole. Terra/GeoPentech’s geologist produced a field log for each boring, with material descriptions logged according to the Unified Soil Classification System. The boring logs are presented in Appendix A.

The borings were backfilled with bentonite pellets (hydrated with water) upon completion.

2.2 TEST PIT EXCAVATION AND BACKFILLING

Three test pits were excavated as part of the field investigation. Each pit was located adjacent to one of the borings drilled on the Main Dam and at Dikes 2 and 5 as shown on Figure 1, approximately along the embankment centerline. TP-202 at the Main Dam was excavated somewhat upstream of centerline to provide more space for stockpiling trench spoil (which was limited due to the roadway guardrails along the crest of the dam), and was therefore excavated along the upstream edge of the core. Consequently, core material was exposed along the downstream (west) wall of the trench, but thinned out in an upstream direction to a point that it was not exposed on the upstream (east) wall of the trench. The test pits were excavated using a JD-310 backhoe with a 2-foot wide bucket, provided and operated by PC Exploration. The excavation work commenced on July 21 and was completed on July 23, 2008.

Each excavation was sloped and benched down to the level of the top of the core (4 to 5 feet deep), to provide for safe access into the trench. This produced an excavation that was 10 to 12-feet wide at the surface, and 15 to 20 feet in length. The lower portion of each trench was excavated approximately 4 feet into the core.

The trench walls within the core zone were scraped to remove smearing, and were closely examined for the presence or absence of desiccation cracking. As discussed below, no indication of cracking was observed in any of the trenches. Terra/GeoPentech’s geologist logged and photographed each trench, and produced a graphic sketch along with material descriptions for each log. The test pit logs are included in Appendix A. Trenches that were left open overnight were safeguarded with stakes, fencing, caution tape and spoil berms.

Annotated photographs of the upper shell and core zone materials as exposed in the three test pits, along with photographs of the excavation and backfilling operations, are contained in Figures 2 through 4 for Dike 2, Figures 5 through 8 for the Main Dam, and Figures 9 through 12 for Dike 5.

The lower core zone-portion of each test pit was backfilled with a lean, controlled low strength concrete (a.k.a. "flowable fill"). The upper portion of the trench was backfilled with the material excavated from the trench (mostly sandy gravel and cobbles, with lesser admixed core zone material consisting of silty sand, sandy silt and clayey sand), that was placed after the underlying concrete backfill had set. The upper backfill was placed and spread with the backhoe in loose lifts of 1 to 1½ -ft. thick, which were then compacted with a hand-operated Ingersoll-Rand vibratory compactor with an 18"x20" plate. PC Exploration provided the equipment, materials and labor to complete the backfilling of the trenches. The Main Dam asphalt roadway at Test Pit TP-202 (at approximately Station 314+25), was saw-cut and removed prior to excavation and then repaved after completion of backfilling and compaction. That paving work was performed by Lodi Grading and Paving under subcontract to PC Exploration.

2.3 UPPER EMBANKMENT SHELL AND CORE CONDITIONS – FIELD OBSERVATIONS

The top of the "Zone 1" core was encountered at depths ranging from approximately 4 to 5 feet at all of the dike and Main Dam boring and trench locations explored for this investigation. The "Zone 6" slope protection that comprises the shell material overlying the core consists of sandy and silty sandy gravel and cobbles, and includes numerous very hard cobbles (mostly quartzite) that are up to 8-inches in diameter.

Field examination of boring samples and test pit exposures indicate that the in-place core has an apparent moisture condition ranging from slightly damp to moist, and is locally very moist. The core materials vary from silt, sandy silt, silty sand, sandy clay, and clayey sand, with non-low plastic fines. Some layers of occasional scattered gravels were observed within the uppermost core (within about one foot of the "Zone 6" contact), but these were well and mixed/encased within the fine-grained core matrix. Penetration resistance data from the borings indicate that the core was well compacted to a stiff to very stiff consistency.

No indications of desiccation cracking, or other open cracks or seams, were noted within the core materials exposed along the walls of any of the three test pits.

Photographs of the core and shell materials exposed in the test pits are shown on the following figures:

- Dike 2 Figures 2 & 3
- Main Dam Figures 6, 7 & 8
- Dike 5 Figures 9 & 10

3.1 PURPOSE AND SCOPE

The purpose of the 2008 Phase 1 laboratory testing program was to evaluate the potential for desiccation cracking in the upper portion of the core of the dam and obtain data on the susceptibility of the core material to internal erosion. As reported in Section 2.3, field examination of boring samples and test pit exposures indicate the in-place core has an apparent moisture of slightly damp to moist and is locally very moist, and there was no indication of desiccation cracking or other open cracks or seams within the ore materials exposed along the walls of test pits TP-201, 202 and 203.

Typically five California samples were driven at 2 ft. intervals at each of the eleven boring locations to obtain samples of the upper 10 feet of the core material at the Main Dam and the six dikes. Each California sample typically provided two samples that were designated as samples A and B.

Appendix B contains the results of the laboratory testing program. Table B-1 provides a summary of the laboratory test results and is followed by detailed results for all tests performed. As shown on Table B-1 the scope of the laboratory testing included:

- Visual classification, moisture content and dry unit weight measurements on all fifty-five samples
- Twenty-Two Atterberg Limit tests; two tests on samples selected from each of the eleven borings
- Eleven determinations of percent passing the No. 200 sieve – one on a sample selected from each boring
- Eleven grain size analyses (sieve and hydrometer) on a sample selected from each boring
- Twenty-Two crumb tests, two tests on samples selected from each of the eleven borings
- Four pinhole erosion tests on a representative sub set of the samples used for the crumb tests

3.2 DISCUSSION OF RESULTS

3.2.1 Percent Fines

Data on the percent passing the No. 200 Sieve are tabulated below. The minimum value for the samples tested was 35.8% of Dike 3 and the maximum values was 95.7% at the Main Dam. The samples tested from the Main Dam have percent fines that are larger than those for samples obtained from the dikes.

Embankment	Percent Passing No. 200 Sieve			
Dike 1	41.8	58.0	70.8	89.8
Dike 2	41.2	47.3	49.0	56.7
Dike 3	35.8	41.1	51.5	54.3

Embankment	Percent Passing No. 200 Sieve			
Main Dam	68.3	73.9	82.2	95.7
Dike 4	44.2	67.3		
Dike 5	38.9	57.7		
Dike 6	43.8	53.6		

3.2.2 Atterberg Limits

Figure 13 shows the results of Atterberg limit tests on the plasticity chart. All the specimens tested are either CL or ML with the liquid limit typically between 20 and 30 and the plasticity index typically between 5 and 12. Four of the 22 samples tested were found to have non-plastic fines.

3.2.3 Moisture Content and Liquidity Index

Figure 14 contains plots of moisture content vs. depth for each of the six dikes and the main dam. Inspection of figure 14 indicates the moisture content at the main dam is typically higher than at the dikes. A trend of increasing water content with depth would be expected if drying of the core materials were occurring due to desiccation. The data do not show such a trend and therefore indicate there has been no change in moisture content within the upper portion of the core since the embankments were constructed.

Figure 15 is a plot of Liquidity Index (LI) vs. depth. LI is obtained by normalizing the moisture content to the Atterberg limits. The LI is 0 when the moisture content is at the plastic limit and the LI is 1.0 when the moisture content is at the liquid limit. The LI data show a relatively large amount of scatter; most likely due to the fact that the plasticity index is low and small changes in moisture content cause large changes in LI. As shown on Figure 15, the liquidity index of the core in the main dam is higher than the LI for the dikes and there is no apparent trend of increasing LI with depth and therefore no indication that drying or the upper portion of the core due to desiccation has occurred.

3.2.4 Dispersivity Tests

The dispersivity of the soils was evaluated using Crumb Tests and Pinhole Dispersion Tests.

A Crumb Test is a relatively simple test used to identify a potentially dispersive soil. Soil crumbs approximately 6 to 10 mm in diameter are prepared from samples of the soil at the natural water content and then dropped into a beaker containing a dilute 0.001 Molar solution of Sodium Hydroxide. The reaction of the soil crumbs is observed for 5 to 10 minutes and the dispersive category of the soil is classified based on the reaction of the soil crumbs.

The samples are graded on the following scale:

- G1 – No Reaction Non-Dispersive
- G2 – Slight Reaction Non-Dispersive

- G3 – Moderate Reaction Dispersive
- G4 – Strong Reaction Dispersive

Appendix B contains data sheets summarizing the laboratory test results

Twenty-two crumb tests were completed, two for each of the eleven borings. The distribution of the test results after 10 minutes of soaking are summarized below:

Embankment	Crumb Test Results
Main Dam	G2, G2, G1, G2
Dike 1	G3, G3, G2, G2
Dike 2	G3, G3, G2, G2
Dike 3	G3, G4, G4, G4
Dike 4	G3, G3
Dike 5	G2, G4
Dike 6	G4, G4

The Pinhole Dispersion Tests were conducted in accordance with ASTM D 4647. This test entails using a pin to pierce a 1-mm diameter hole in a 1.5-inch long test specimen, and then measuring the flow rate through the pinhole and observing the cloudiness of the effluent under hydraulic heads of 2, 7, 15, and 40 inches. The dispersive classification is assigned based on the following criteria:

Dispersive Classification	Head (in)	Test time at constant head (min)	Final flow through specimen (mL/s)	Cloudiness of flow at the end of the test		Hole size after test (mm)
				From side	From top	
D1	2	5	1.0 to 1.4	Dark	Very dark	≥ 2.0
D2	2	10	1.0 to 1.4	Moderately dark	Dark	> 1.5
ND4	2	10	0.8 to 1.0	Slightly dark	Moderately dark	≤ 1.5
ND3	7	5	1.4 to 2.7	Barely visible	Slightly dark	≥ 1.5
	15	5	1.8 to 3.2	Barely visible	Slightly dark	≥ 1.5

Dispersive Classification	Head (in)	Test time at constant head (min)	Final flow through specimen (mL/s)	Cloudiness of flow at the end of the test		Hole size after test (mm)
				From side	From top	
ND2	40	5	> 3.0	Clear	Barely visible	< 1.5
ND1	40	5	≤ 3.0	Perfectly clear	Perfectly clear	1.0

Four pinhole tests were run and all resulted in a dispersive classification of ND1 – the least dispersive classification possible. The samples used for the Pinhole Dispersion Tests and corresponding results of Crumb Tests are summarized below:

Embankment	Boring No.	Sample No.	Depth, ft	Crumb Test	Pinhole Test
Main Dam	DH-207	MC-2	7.25	G2	ND1
Dike 2	DH-203	MC-2A	7.25	G3	ND1
Dike 3	DH-206	MC-5	13.25	G4	ND1
Dike 6	DH-211	MC-3A	9.25	G4	ND1

Based on these results we have concluded that the Crumb Tests indicate that the soil has the potential for being dispersive but that the Pinhole Dispersion tests show that the soils are indeed not dispersive or erodible when subjected to a concentrated flow of water. Consequently, the soils are classified as non-dispersive for the purpose of our evaluation of piping potential.

FIGURES

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LOCATIONS OF BORINGS AND TEST PITS
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure 1

Rev. 4 10/24/08

NORTH WALL: ZONE 6 SLOPE PROTECTION SHELL ON ZONE 1 CORE

Rev. 1 10/14/08

TERRA / GeoPentech a Joint Venture	DIKE 2 - TP 201 (1 OF 3) CAMANCHE EMBANKMENTS SAFETY REVIEW	Figure 2

NORTH WALL: CLOSE-UP VIEW OF SHELL/CORE CONTACT

TERRA / GeoPentech a Joint Venture	DIKE 2 - TP 201 (2 OF 3) CAMANCHE EMBANKMENTS SAFETY REVIEW	Figure 3

Rev. 1 10/14/08

COMPACTION OF ZONE 6 SLOPE PROTECTION BACKFILL

Rev. 1 10/14/08

TERRA / GeoPentech a Joint Venture	DIKE 2 - TP 201 (3 OF 3) CAMANCHE EMBANKMENTS SAFETY REVIEW	Figure 4

WEST WALL: ZONE 6 SLOPE PROTECTION SHELL ON ZONE 1 CORE; HAMMER AT TOP OF CORE

TERRA / GeoPentech a Joint Venture	MAIN DAM - TP 202 (1 OF 4) CAMANCHE EMBANKMENTS SAFETY REVIEW	Figure 5

Rev. 1 10/14/08

WEST WALL AND NORTH END: ZONE 6 SLOPE PROTECTION SHELL ON ZONE 1 CORE; HAMMER AT TOP OF CORE; TRENCH LOCATED ALONG UPSTREAM EDGE OF CORE; HENCE THINNING CORE AND THICKENING GRAVEL LAYERS TOWARD UPSTREAM (RIGHT)

TERRA / GeoPentech a Joint Venture	MAIN DAM - TP 202 (2 OF 4) CAMANCHE EMBANKMENTS SAFETY REVIEW	Figure 6

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WEST WALL: CLOSE UP VIEW OF ZONE 1 CORE

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MAIN DAM - TP 202
(3 OF 4)
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
7

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REPAVING MAIN DAM ROADWAY
AT TP 202

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MAIN DAM - TP 202
(4 OF 4)
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
8

Rev. # 10/14/08

EAST WALL: ZONE 6 SLOPE PROTECTION ON ZONE 1 CORE

EAST WALL: CLOSE UP VIEW OF ZONE 1 CORE

Rev. 1 10/14/08

TERRA / GeoPentech a Joint Venture	DIKE 5 - TP 203 (2 OF 4) CAMANCHE EMBANKMENTS SAFETY REVIEW	Figure 10

CONCRETE BACKFILL OF ZONE 1 CORE

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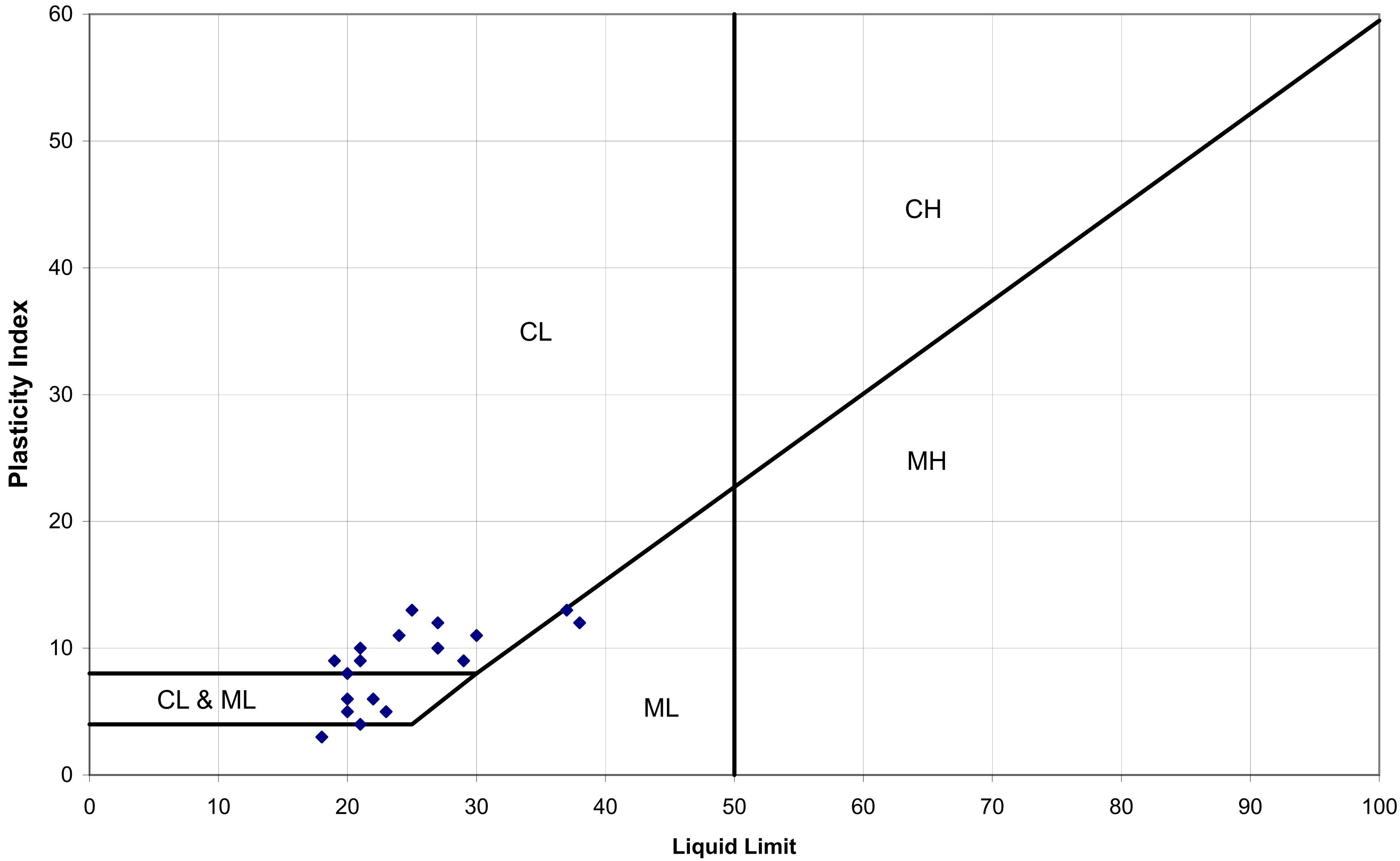
DIKE 5 - TP 203
(3 OF 4)
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
11

PLACING INITIAL LIFT OF ZONE 6 SHELL BACKFILL OVER CONCRETE BACKFILL



4 OF 22 SAMPLES HAD
NON-PLASTIC FINES

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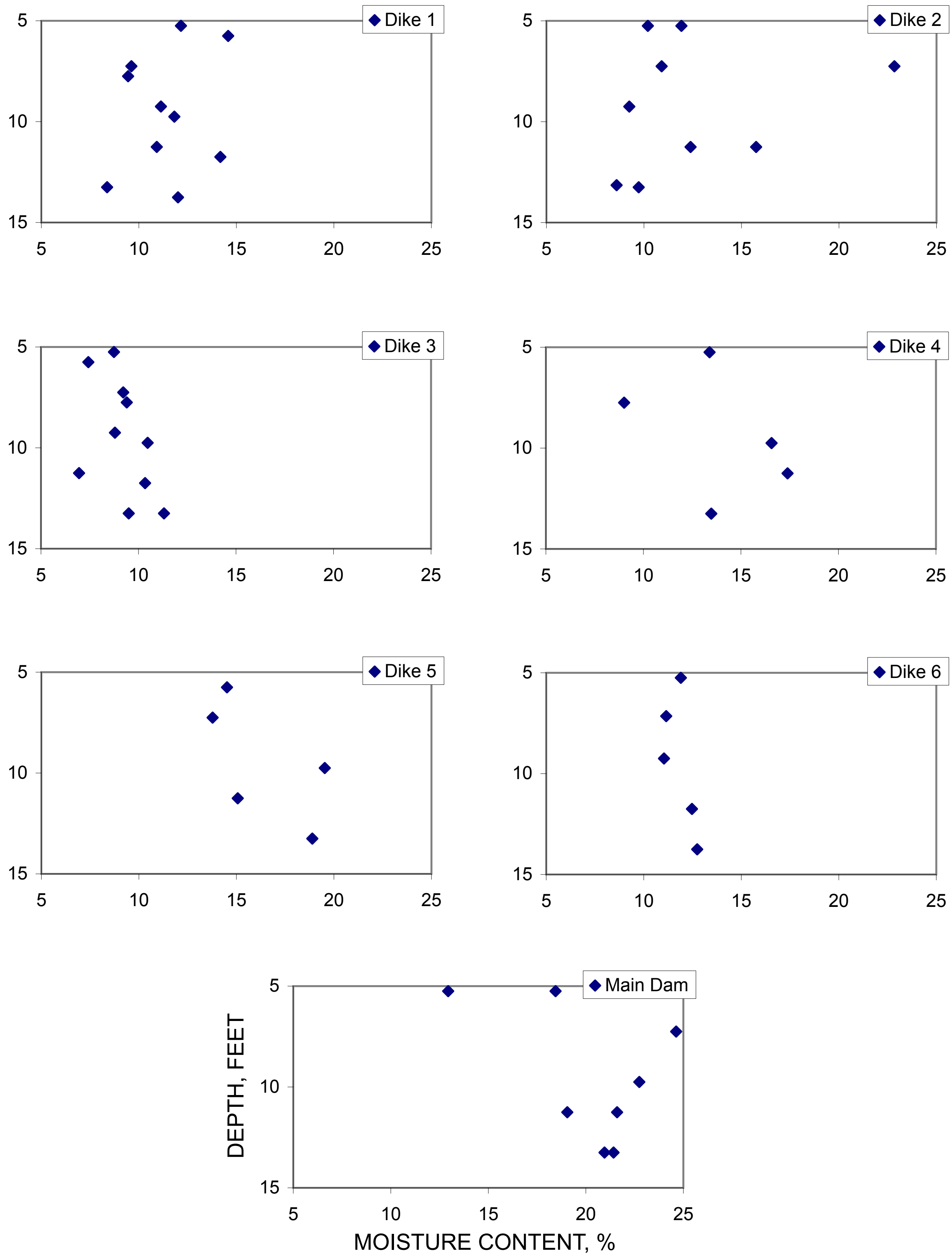
PLASTICITY CHART

CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
13

Rev. 1 10/16/08



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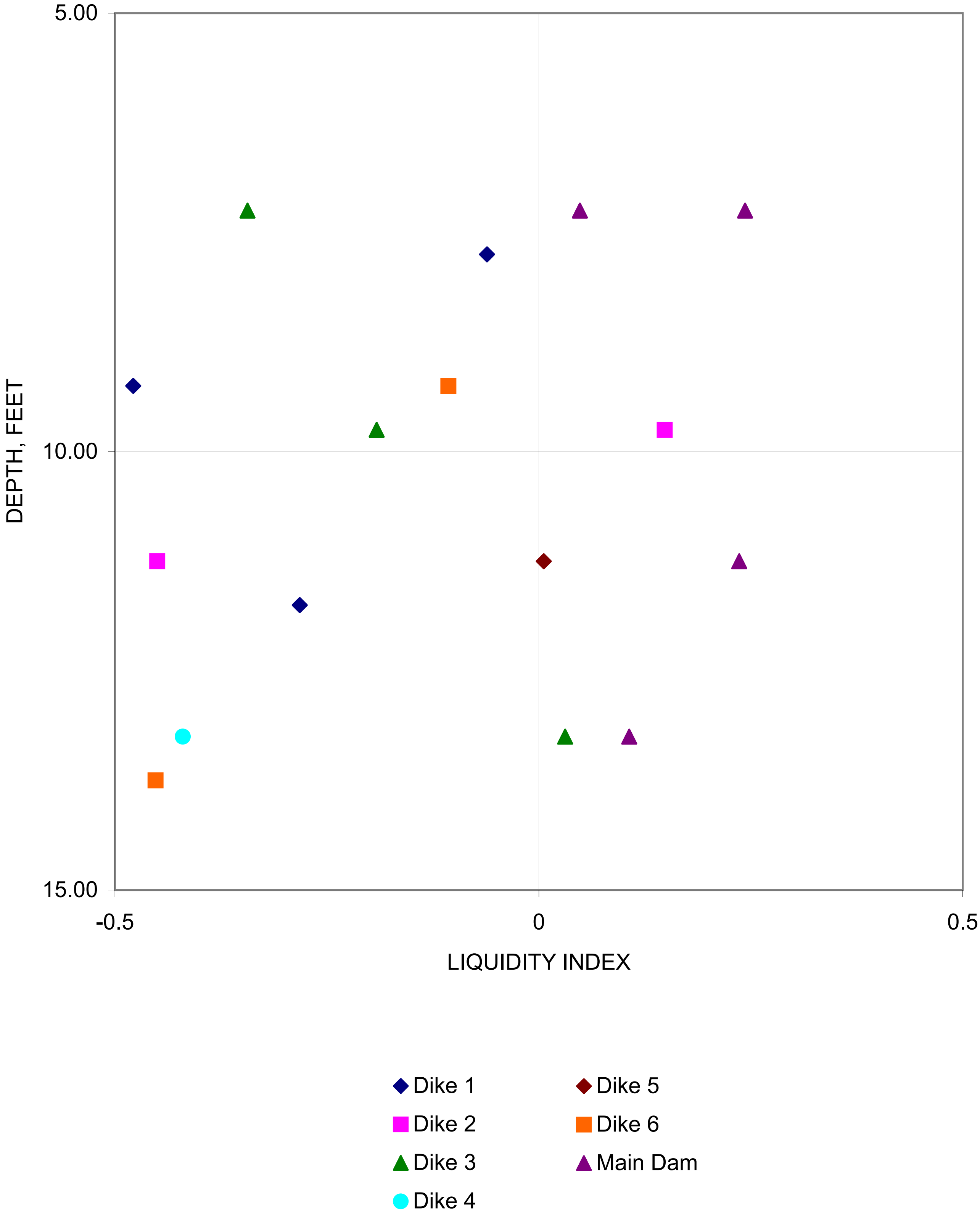
MOISTURE CONTENT
VS DEPTH
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure
14



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LIQUDITY INDEX
VS DEPTH
CAMANCHE EMBANKMENTS SAFETY REVIEW

Figure 15

Rev. 1 10/16/08

APPENDIX A

CAMANCHE EMBANKMENTS SAFETY REVIEW	Key to Log of Boring Sheet 1 of 1

Elevation, feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	SAMPLES: Blows Per 6 inches	SAMPLES: Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15

COLUMN DESCRIPTIONS

1 **Elevation:** Elevation in feet referenced to specified datum.

2 **Depth:** Depth in feet below the ground surface.

3 **Sample Type:** Type of soil or rock sample collected at depth interval shown; sampler symbols are explained below.

4 **Sample Number:** Sample identification number.

5 **Blows per 6 inches:** Number of blows required to advance sampler each 6-inch drive interval, or distance noted, using a 140-lb hammer dropped 30 inches.

6 **Recovery:** Length in inches of material recovered in sampler.

7 **Graphic Log:** Graphic depiction of subsurface material encountered. Typical symbols are given below; variations on these symbols are used to indicate secondary soil components.

8 **Material Description:** Description of material encountered; in addition to soil or rock classification, may include color, moisture, relative density/consistency, particle size, and plasticity for soil; texture, weathering, strength, and hardness of bedrock.

9 **% Gravel:** Percent of soil by weight retained on the No. 4 sieve as determined per ASTM Method D422.

10 **% Fines:** Percent of soil by weight passing the No. 200 sieve as determined per ASTM Method D422.

11 **Liquid Limit:** Liquid Limit (LL) of soil specimen passing the No. 40 sieve as determined per ASTM Method D4318.

12 **Plasticity Index:** Plasticity Index (PI=LL-PL) of soil specimen passing the No. 40 sieve as determined per ASTM Method D4318.

13 **Moisture Content:** Moisture content, as a percentage of dry weight of specimen, determined per ASTM Method D2216.

14 **Dry Unit Weight:** Dry weight per unit volume of soil, reported in pounds per cubic foot, determined per ASTM Method D2937.

15 **Field Notes and Other Tests:** Comments and observations regarding drilling or sampling made by driller or field personnel. Lab test results other than those listed in columnar format may be recorded using abbreviations below.

TYPICAL MATERIAL GRAPHIC SYMBOLS

- POORLY GRADED SAND (SP)
- WELL-GRADED SAND with SILT (SW-SM)
- SILTY SAND (SM)
- CLAYEY SAND (SC)
- SILT (ML)
- ELASTIC SILT (MH)
- SANDY SILT (ML)
- SILTY, CLAYEY SAND (SC-SM)
- LEAN CLAY (CL)
- FAT CLAY (CH)
- SANDY LEAN CLAY (CL)
- SILTY CLAY (CL-ML)
- POORLY GRADED GRAVEL (GP)
- WELL-GRADED GRAVEL (GW)
- SILTY GRAVEL (GM)
- CLAYEY GRAVEL (GC)

TYPICAL SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT) split spoon
- Shelby tube (thin-wall, fixed-head undisturbed)
- Pitcher Barrel (lined with Shelby tube)
- HQ rock core barrel
- Modified California (2.4-inch-ID, 3-inch-OD)
- Bulk sample collected from auger cuttings

OTHER LABORATORY TEST ABBREVIATIONS

CONS One-Dimensional Consolidation Test
CRUMB Crumb Test
TX-UU Unconsolidated Undrained Triaxial Compression Test
TX-CIU Isotropically Consolidated Undrained Triaxial Test
UCS Unconsolidated Compressive Strength Test
%<5μ Hydrometer Test (coupled with Sieve Test)

OTHER GRAPHIC SYMBOLS

- First water encountered at time of drilling and sampling
- Static water level measured at specified time after drilling
- Change in material properties within a lithologic unit
- Inferred contact between soil strata or gradational change

GENERAL NOTES

1. Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.

2. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

Report: TGP SOIL+LAB KEY; File: TGP_CAMANCHE.GPJ; 8/24/2008

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CAMANCHE EMBANKMENTS SAFETY REVIEW	Log of Boring DH-201 Sheet 1 of 1

Date(s) Drilled	**7/16/08**	Logged By	**R. Harlan**	Checked By	
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**6-inch-OD auger**	Total Depth of Borehole	**14.5 feet**
Drill Rig Type	**Ingersoll Rand A400**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Dry at time of drilling**	Sampling Method	**Modified California (2.4-inch-ID)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Dike 1 crest, 10 feet U/S of D/S edge, about 1/3 of crest length W of E end**	Borehole Completion	**Hydrated bentonite chips, gravel cuttings top 1 foot**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
265	0						**CREST / SLOPE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SAND (GW), moderate yellowish brown, dry, fine to coarse gravel to 3 inches, fine- to coarse-grained sand, some cobbles							Start drilling at 1025.
							SILTY GRAVEL with SAND (GM), yellowish gray, dry, mostly fine rounded gravel, minor coarse gravel, medium- to coarse-grained sand							
260	5		MC-1A MC-1B	27 35 58	18		**CORE (ZONE 1)** SANDY SILT (ML), greenish gray to 5.5 feet, dark brown below, slightly damp to damp, nonplastic fines, mostly fine-grained sand					14.6	107	Smooth drilling at 5 ft.
			MC-2A MC-2B	13 18 18	18		CLAYEY SAND (SC), dark brown, slightly damp to damp, fine- to medium-grained sand, low plasticity fines	0	42	19	9	9.5	132	20%<5μ; CRUMB
	10		MC-3A MC-3B	13 25 43	18		Becomes moderate yellowish brown					11.8	121	
255			MC-4A MC-4B	17 20 35	18		LEAN CLAY with SAND (CL), dark brown, damp, low plasticity fines, mostly fine-grained sand		71	27	10	14.2	119	
			MC-5A MC-5B	15 18 23	18		SILTY SAND (SM), dark yellowish brown to orange brown, damp, fine-grained sand, low plasticity fines					12.0	109	
250	15						**Bottom of boring at 14.5 feet**							Finish drilling at 1200; backfill hole.
245	20													
	25													

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CAMANCHE EMBANKMENTS SAFETY REVIEW

Log of Boring DH-202

Date(s) Drilled	**7/16/08**	Logged By	**R. Harlan**	Checked By	
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**6-inch-OD auger**	Total Depth of Borehole	**14.5 feet**
Drill Rig Type	**Ingersoll Rand A400**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Dry at time of drilling**	Sampling Method	**Modified California (2.4-inch-ID)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Dike 1 crest, 10 feet U/S of D/S edge, about 1/3 of crest length E of W end**	Borehole Completion	**Hydrated bentonite chips, sand and gravel cuttings top 1 foot**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
	0						**CREST / SLOPE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SAND (GW), moderate yellowish brown, dry, fine to coarse gravel to 3 inches, fine- to coarse-grained sand, some cobbles							Start drilling at 1305.
265														
							CORE (ZONE 1) CLAYEY GRAVEL (GC), dark yellowish brown, damp, low to medium plasticity fines							Smooth drilling at 4 ft.
	5		MC-1A	21								12.2	99	
260			MC-1B	21 22	13		SANDY SILT (ML), yellowish brown and dark brown, damp, low plasticity fines, mostly fine-grained sand, minor medium- to coarse-grained sand, includes occasional pockets/layers of LEAN CLAY (CL), gray, low to medium plasticity							
			MC-2A	6								9.6	118	
			MC-2B	13 25	14		← Light yellowish brown seam							
			MC-3A	10					58	20	6	11.1	112	CRUMB
	10		MC-3B	25 34	14		SILT (ML), dark grayish brown, slightly damp, nonplastic to low plasticity, trace fine-grained sand							
255			MC-4A	14								10.9	112	
			MC-4B	22 43	14									
			MC-5	31 30 30	5		Moderate yellowish brown layer	0	90	NP	NP	8.4	129	12%<5μ; CRUMB
	15						**Bottom of boring at 14.5 feet**							Finish drilling at 1430; backfill hole.
250														
	20													
245														
	25													

Report: TGP SOIL+LAB; File: TGP_CAMANCHE.GPJ; 8/24/2008

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CAMANCHE EMBANKMENTS SAFETY REVIEW	Log of Boring DH-203 Sheet 1 of 1

Date(s) Drilled	**7/16/08**	Logged By	**R. Harlan**	Checked By	
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**6-inch-OD auger**	Total Depth of Borehole	**14.0 feet**
Drill Rig Type	**Ingersoll Rand A400**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Dry at time of drilling**	Sampling Method	**Modified California (2.4-inch-ID)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Dike 2 crest, 10 feet U/S of D/S edge, about 1/3 of crest length W of E end**	Borehole Completion	**Hydrated bentonite chips, sand and gravel cuttings top 1 foot**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
265	0						**CREST / SLOPE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SAND (GW), moderate yellowish brown, dry, fine to coarse gravel to 3 inches, fine- to coarse-grained sand, some cobbles							Start drilling at 1510.
260	5		MC-1A MC-1B	15 27 39	10		**CORE (ZONE 1)** SANDY LEAN CLAY (CL), moderate yellowish brown, damp, low plasticity fines, fine-grained sand					10.2	126	Smooth drilling at 4.5 ft.
			MC-2A MC-2B	28 22 50	14		SILTY, CLAYEY SAND (SC-SM), dark gray and dark grayish brown with bluish gray pockets/layers, slightly damp to damp, mostly fine-grained sand, minor medium-grained sand, low plasticity fines	0	47	21	4	10.9	122	16%<5μ; CRUMB
	10		MC-3	10 16 21	5		CLAYEY SAND (SC), dark yellowish brown, damp, fine-grained sand, low plasticity fines					9.3	126	
255			MC-4	16 18 23	5		SANDY SILT (ML), light yellowish brown, damp, low plasticity fines, fine-grained sand, includes pockets of CLAYEY SAND to SANDY LEAN CLAY (SC/CL)		49	23	5	15.8	106	
			MC-5A MC-5B	20 75/6"	9		Becomes dark grayish brown with dark yellowish brown clayey pockets, slightly damp, nonplastic to low plasticity fines					8.6	132	Mod Cal sampler refusal at 14 ft.
							Bottom of boring at 14.0 feet							Finish drilling at 1625; backfill hole.
250	15													
245	20													
	25													

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CAMANCHE EMBANKMENTS SAFETY REVIEW	Log of Boring DH-204 Sheet 1 of 1

Date(s) Drilled	**7/17/08**	Logged By	**R. Harlan**	Checked By	
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**6-inch-OD auger**	Total Depth of Borehole	**14.5 feet**
Drill Rig Type	**Ingersoll Rand A400**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Dry at time of drilling**	Sampling Method	**Modified California (2.4-inch-ID)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Dike 2 crest, 10 feet U/S of D/S edge, about 1/3 of crest length E of W end**	Borehole Completion	**Hydrated bentonite chips, sand and gravel cuttings top 1 foot**		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows Per 6 inches	SAMPLES Recovery, inches	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
265	0					**CREST / SLOPE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SAND (GW), moderate yellowish brown, dry, fine to coarse gravel to 3 inches, fine- to coarse-grained sand, some cobbles							Start drilling at 0800.
						SILTY SAND to SANDY SILT (SM/ML)							Smooth drilling 3.5-4.5 ft. Rough gravelly drilling 4.5-5 ft. Smooth drilling at 5 ft.
260	5	MC	MC-1	14 18 18	6	**CORE (ZONE 1)** CLAYEY SAND (SC), moderate to dark yellowish brown, damp to moist, mostly fine-grained sand, minor medium- to coarse-grained sand, low to medium plasticity fines					11.9	111	
		MC	MC-2	10 19 25	7	SANDY LEAN CLAY (CL), moderate to dark yellowish brown, damp to moist, low plasticity fines, mostly fine-grained sand, minor coarse-grained sand					22.8	99	
						CLAYEY SAND (SC), dark reddish brown, moist, fine-grained sand, low to medium plasticity fines							
	10	MC	MC-3A MC-3B	9 22 34	18	SANDY LEAN CLAY (CL), dark gray, damp, low plasticity fines, fine-grained sand		57	25	13	13.9	114	CRUMB
255		MC	MC-4A MC-4B	15 23 36	15	Becomes dark yellowish brown					12.4	121	
		MC	MC-5A MC-5B	17 28 24	15	SILTY SAND (SM), dark yellowish brown and reddish brown, damp to moist, fine- to medium-grained sand, low plasticity fines	2	41	18	3	9.7	123	12%<5μ; CRUMB
250	15					**Bottom of boring at 14.5 feet**							Finish drilling at 0908; backfill hole. Note: Bulk cuttings appear moister than at DH-201, DH-202, and DH-203. Hole located at left end of buttressed portion of Dike 2 over deep alluvial foundation.
245	20												
	25												

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CAMANCHE EMBANKMENTS SAFETY REVIEW	Log of Boring DH-205 Sheet 1 of 1

Date(s) Drilled	**7/17/08**	Logged By	**R. Harlan**	Checked By	
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**6-inch-OD auger**	Total Depth of Borehole	**14.5 feet**
Drill Rig Type	**Ingersoll Rand A400**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Dry at time of drilling**	Sampling Method	**Modified California (2.4-inch-ID)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Dike 3 crest, 12 feet U/S of D/S edge, about 1/3 crest length NW of SE end**	Borehole Completion	**Hydrated bentonite chips, sand and gravel cuttings top 1 foot, asphalt patch**		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows Per 6 inches	SAMPLES Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
	0						Asphalt							Start drilling at 0930.
265							**CREST / SLOPE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SAND (GW), moderate yellowish brown, dry, fine to coarse gravel to 3 inches, fine- to coarse-grained sand, some cobbles							
	5													
260			MC-1	46 43 40	8		**CORE (ZONE 1)** CLAYEY GRAVEL (GC), moderate yellowish brown, slightly damp; over SILTY SAND (SM), dark gray, damp, fine-grained sand, nonplastic to low plasticity fines					7.4	118	Smooth drilling at 5.5 ft.
			MC-2A MC-2B	23 43 57	15		CLAYEY SAND with GRAVEL (SC), dark yellowish brown, damp, fine- to medium-grained sand, low plasticity fines	18	41	24	11	9.2	124	16%<5μ; CRUMB
							SANDY SILT (ML), moderate yellowish brown, slightly damp to damp, nonplastic to low plasticity fines, fine-grained sand							
			MC-3A MC-3B	38 70/6"	12		CLAYEY SAND to SANDY LEAN CLAY (SC/CL), dark yellowish brown, slightly damp to damp					8.8	124	
	10													
255							SILTY SAND (SM), dark grayish brown, slightly damp to damp, fine-grained sand, nonplastic fines							
			MC-4	27 40 26	9				36	NP	NP	7.0	133	CRUMB
			MC-5A MC-5B	17 27 28	14		Becomes damp					9.5	127	
							CLAYEY SAND (SC), dark reddish brown, damp, fine-grained sand, low to medium plasticity fines							
	15						**Bottom of boring at 14.5 feet**							Finish drilling at 1045; backfill hole.
250														
	20													
245														
	25													

CAMANCHE EMBANKMENTS SAFETY REVIEW	Log of Boring DH-206 Sheet 1 of 1

Date(s) Drilled	**7/17/08**	Logged By	**R. Harlan**	Checked By	
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**6-inch-OD auger**	Total Depth of Borehole	**14.5 feet**
Drill Rig Type	**Ingersoll Rand A400**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Dry at time of drilling**	Sampling Method	**Modified California (2.4-inch-ID)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Dike 3 crest, 11 feet U/S of D/S edge, about 300 feet S of left spillway wall**	Borehole Completion	**Hydrated bentonite chips, sand and gravel cuttings top 1 foot**		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows Per 6 inches	SAMPLES Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
	0						**CREST / SLOPE PROTECTION (ZONE 6)**							Start drilling at 1115.
265							WELL-GRADED GRAVEL with SAND (GW), moderate yellowish brown, dry, fine to coarse gravel to 3 inches, fine- to coarse-grained sand, some cobbles							
							CORE (ZONE 1)							Smooth drilling at 4.5 ft.
	5		MC-1A MC-1B	25 23 47	14		CLAYEY GRAVEL (GC), moderate yellowish brown, slightly damp; over SILTY SAND (SM), dark grayish brown, damp					8.7	125	
260							SANDY SILT (ML), dark grayish brown, slightly damp, low plasticity fines, fine-grained sand							
			MC-2A MC-2B	18 36 28	18		SILTY SAND (SM), moderate yellowish brown, damp, fine-grained sand, nonplastic to low plasticity fines					9.4	129	
	10		MC-3A MC-3B	15 16 17	17		SANDY LEAN CLAY (CL), dark yellowish brown, slightly damp, low plasticity fines, fine- to medium-grained sand Becomes dark grayish brown, damp		54	20	8	10.5	122	CRUMB
255			MC-4A MC-4B	10 27 45	18		Becomes mixed dark yellowish brown and grayish brown with pockets of greenish gray					10.3	122	
			MC-5	7 23 29	9			0	52	21	10	11.3	120	19%<5μ; CRUMB
	15						**Bottom of boring at 14.5 feet**							Finish drilling at 1220; backfill hole.
250														
	20													
245														
	25													

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Log of Boring DH-207

Sheet 1 of 1

Date(s) Drilled	**7/17/08**	Logged By	**R. Harlan**	Checked By	
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**6-inch-OD auger**	Total Depth of Borehole	**14.5 feet**
Drill Rig Type	**Ingersoll Rand A400**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Dry at time of drilling**	Sampling Method	**Modified California (2.4-inch-ID)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Main Dam crest, 17 feet U/S of D/S edge, approx. Station 314+25**	Borehole Completion	**Hydrated bentonite chips, sand and gravel cuttings top 1 foot, asphalt patch**		

Elevation, feet	Depth, feet	Sample Type	Sample Number	Blows Per 6 inches	Recovery, inches	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
	0					Asphalt							Start drilling at 1255.
265						**CREST / SLOPE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SAND (GW), moderate yellowish brown, dry, fine to coarse gravel to 3 inches, fine- to coarse-grained sand, some cobbles							
260	5	MC	MC-1	15 33 25	8	**CORE (ZONE 1)** CLAYEY GRAVEL (GC), moderate yellowish brown, damp to moist, gravel to 1 inch, low to medium plasticity fines					12.9	107	Smooth drilling at 5 ft.
		MC	MC-2	35 47 25	9	SILT with SAND (ML), moderate yellowish brown, moist, low plasticity fines, fine- to medium-grained sand	0	82	38	12	28.9	91	26%<5μ; CRUMB
	10	MC	MC-3	12 27 30	9	Becomes damp					22.7	101	
255						SANDY LEAN CLAY (CL), moderate yellowish brown, damp, low to medium plasticity, fine-grained sand; Few gravel							
		MC	MC-4	15 30 45	8	SANDY SILT (ML), dark gray, damp, nonplastic to low plasticity fines, fine-grained sand, includes pockets/layers of moderate yellowish brown silt to gravelly silt		68	30	11	21.6	100	CRUMB
		MC	MC-5A MC-5B	11 34 41	10	Becomes moderate yellowish brown, low plasticity fines					21.4	101	
	15					**Bottom of boring at 14.5 feet**							Finish drilling at 1345; backfill hole.
250													
	20												
245													
	25												

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CAMANCHE EMBANKMENTS SAFETY REVIEW

Log of Boring DH-208

Sheet 1 of 1

Date(s) Drilled	**7/17/08**	Logged By	**R. Harlan**	Checked By	
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**6-inch-OD auger**	Total Depth of Borehole	**16.5 feet**
Drill Rig Type	**Ingersoll Rand A400**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Dry at time of drilling**	Sampling Method	**Modified California (2.4-inch-ID)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Main Dam crest, 17 feet U/S of D/S edge, approx. Station 322+10**	Borehole Completion	**Hydrated bentonite chips, sand and gravel cuttings top 1 foot, asphalt patch**		

Elevation, feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	SAMPLES: Blows Per 6 inches	SAMPLES: Recovery, inches	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
	0					Asphalt							Start drilling at 1410.
265						**CREST / SLOPE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SAND (GW), moderate yellowish brown, dry, fine to coarse gravel to 3 inches, fine- to coarse-grained sand, some cobbles							
	5	MC	MC-1	16 35 41	8	**CORE (ZONE 1)** SANDY SILT (ML), moderate yellowish brown, moist, nonplastic fines, fine-grained sand					18.5	101	Smooth drilling at 5 ft.
260		MC	MC-2	15 23 38	7	LEAN CLAY (CL), moderate yellowish brown, damp to moist, low to medium plasticity, trace fine-grained sand, includes occasional pockets/layers of dark gray SILT to SANDY SILT (ML)	0	96	37	13	24.6	98	24%<5μ; CRUMB
		MC		6 16 18	0								No recovery at 9 ft.
255	10	MC	MC-3A MC-3B	15 20 27	12	SANDY SILT (ML), dark gray, damp, nonplastic to low plasticity fines, fine-grained sand					19.1	84	Install catcher in sampler at 11 ft.
		MC	MC-4	11 12 40	8	LEAN CLAY with SAND (CL), mixed moderate yellowish brown and greenish gray, damp, low plasticity fines, fine-grained sand		74	29	9	21.0	109	CRUMB
250	15	MC	MC-5A MC-5B	6 29 50	13	SANDY SILT (ML), dark brown, damp, nonplastic to low plasticity fines, fine-grained sand					12.5	124	
						Bottom of boring at 16.5 feet							Finish drilling at 1515; backfill hole.
245	20												
	25												

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Log of Boring DH-209

Date(s) Drilled	**7/18/08**	Logged By	**R. Harlan**	Checked By	
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**6-inch-OD auger**	Total Depth of Borehole	**14.5 feet**
Drill Rig Type	**Ingersoll Rand A400**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Dry at time of drilling**	Sampling Method	**Modified California (2.4-inch-ID)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Dike 4 crest, 12 feet U/S of D/S edge, about midpoint of dike**	Borehole Completion	**Hydrated bentonite chips, sand and gravel cuttings top 1 foot**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
265	0						**CREST / SLOPE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SAND (GW), moderate yellowish brown, dry, fine to coarse gravel to 3 inches, fine- to coarse-grained sand, some cobbles							Start drilling at 1050.
260	5		MC-1	13 34 28	8		**CORE (ZONE 1)** SANDY SILT (ML), dark brown and dark grayish brown, damp, low plasticity fines, mostly fine-grained sand					13.4	119	Smooth drilling at 5 ft.
							SILTY SAND (SM), moderate yellowish brown, damp, fine-grained sand, nonplastic fines							
			MC-2	30 30 29	9		Sand grades slightly coarser-grained					9.0	119	
	10		MC-3	12 16 17	10		Wet (perched water) Becomes damp to moist		44	NP	NP	16.6	112	CRUMB
255			MC-4A MC-4B	11 19 30	14		SANDY SILTY CLAY (CL-ML), dark yellowish brown, damp, low plasticity fines, mostly fine-grained sand					17.4	107	
			MC-5A MC-5B	7 17 27	14			0	67	22	6	13.5	114	18%<5μ; CRUMB
	15						**Bottom of boring at 14.5 feet**							Finish drilling at 1130; backfill hole.
250														
	20													
245														
	25													

CAMANCHE EMBANKMENTS SAFETY REVIEW

Log of Boring DH-210

Sheet 1 of 1

Date(s) Drilled	**7/18/08**	Logged By	**R. Harlan**	Checked By	
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**6-inch-OD auger**	Total Depth of Borehole	**14.5 feet**
Drill Rig Type	**Ingersoll Rand A400**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Dry at time of drilling**	Sampling Method	**Modified California (2.4-inch-ID)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Dike 5 crest, 10 feet U/S of D/S edge, about 500 feet N of S end (midpoint)**	Borehole Completion	**Hydrated bentonite chips, sand and gravel cuttings top 1 foot**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
	0						**CREST / SLOPE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SAND (GW), moderate yellowish brown, dry, fine to coarse gravel to 3 inches, fine- to coarse-grained sand, some cobbles							Start drilling at 0920.
265														
							CORE (ZONE 1) SILTY SAND (SM), dark yellowish brown, damp to moist, fine-grained sand, nonplastic to low plasticity fines							Smooth drilling at 4 ft.
	5													
260			MC-1	23 18 28	9			0	39	NP	NP	14.5	113	12%<5μ; CRUMB
			MC-2A MC-2B	22 20 19	14		←Several 1/2 inch-diameter dark gray clay pockets					13.8	117	
							SANDY LEAN CLAY (CL), moderate brown, moist, low to medium plasticity fines, mostly fine-grained sand, minor medium- to coarse-grained sand							
	10		MC-3	16 18 25	10							19.5	105	
255			MC-4A MC-4B	9 15 25	12				58	27	12	15.1	110	CRUMB
							SANDY SILTY CLAY (CL-ML), dark yellowish brown and grayish brown, damp to moist, low plasticity fines, mostly fine-grained sand, minor medium- to coarse-grained sand							
			MC-5A MC-5B	10 30 33	14		Includes pockets of LEAN CLAY (CL), dark grayish brown, moist, medium plasticity					18.9	109	
	15						**Bottom of boring at 14.5 feet**							Finish drilling at 1005; backfill hole.
250														
	20													
245														
	25													

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Log of Boring DH-211

Sheet 1 of 1

Date(s) Drilled	**7/18/08**	Logged By	**R. Harlan**	Checked By	
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**6-inch-OD auger**	Total Depth of Borehole	**14.5 feet**
Drill Rig Type	**Ingersoll Rand A400**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**266 feet MSL**
Groundwater Level(s)	**Dry at time of drilling**	Sampling Method	**Modified California (2.4-inch-ID)**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Dike 6 crest, 12 feet U/S of D/S edge, about 450 feet NE of left end of dike**	Borehole Completion	**Hydrated bentonite chips, sand and gravel cuttings top 1 foot**		

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
265	0						**CREST / SLOPE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SAND (GW), moderate yellowish brown, dry, fine to coarse gravel to 3 inches, fine- to coarse-grained sand, some cobbles							Start drilling at 0800.
260	5	MC	MC-1A MC-1B	22 29 55	14		**CORE (ZONE 1)** SANDY SILT (ML), dark yellowish brown, damp, low plasticity fines, mostly fine-grained sand, minor medium- to coarse-grained sand and gravel to 1 inch					11.9	110	Smooth drilling at 4.5 ft.
		MC	MC-2	40 60/6"	3							11.2	125	Mod Cal refusal at 8 ft; 2-1/2-inch cobble fragment wedged in sampler shoe.
	10	MC	MC-3A MC-3B	12 28 30	12		CLAYEY SAND (SC) and SANDY LEAN CLAY (CL), dark yellowish brown, damp, low plasticity fines, fine-grained sand		54	21	9	11.0	124	CRUMB
255		MC	MC-4A MC-4B	15 20 24	18							12.5	119	
		MC	MC-5A MC-5B	8 11 24	18		SILTY, CLAYEY SAND (SC-SM), dark yellowish brown, damp, fine-grained sand, low plasticity fines Becomes dark gray	0	44	20	5	12.7	121	19%<5μ; CRUMB
250	15						**Bottom of boring at 14.5 feet**							Finish drilling at 0900; backfill hole.
245	20													
	25													

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CAMANCHE EMBANKMENTS SAFETY REVIEW

Log of Test Pit TP-201

Date(s) Excavated	**7/21/08**	Surface Condition	**Gravel roadway along dike crest**	logged By	**R. Harlan**	Checked By	
Excavation Contractor	**PC Exploration**	Weather Condition	**Clear**	Length of Excavation	**20 feet**	Total Depth of Excavation	**9.0 feet**
Excavation Equipment	**JD-310 Backhoe**	Groundwater Observations	**Dry at time of excavation**	Width of Excavation	**12 feet at surface, 2 feet through core**	Approximate Surface Elevation	**266 feet MSL**
Test Pit Location	**Dike 2 crest, about 1/3 of crest length west of east end**			Test Pit Completion	**Core zone portion backfilled with concrete; crest/slope protection gravel backfilled and compacted in approx. 1-ft lifts with vibratory compactor**		

MATERIAL DESCRIPTION

CREST / SLOPE PROTECTION (ZONE 6)

(1) 0 - 4 ft: WELL-GRADED GRAVEL and COBBLES with SAND (GW) to SILTY GRAVEL and COBBLES with SAND (GM), moderate yellowish brown, dry to 2 ft, damp below 2 ft, come cobbles to 6 inches dia.

(2) 4 - 4.8 ft: CLAYEY GRAVEL (GC), moderate yellowish brown, damp to locally very moist (moisture appears to pond on core), low to medium plasticity fines

CORE (ZONE 1)

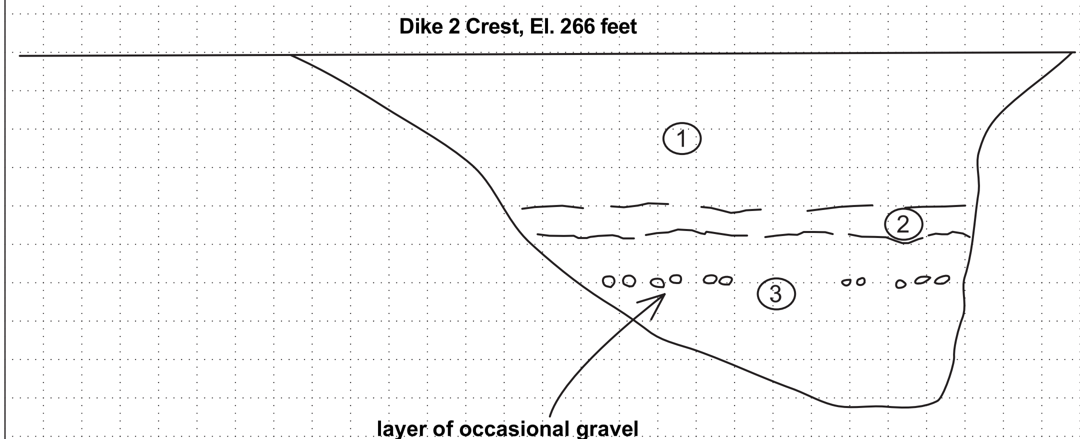
(3) 4.8 - 9 ft: SANDY SILT (MT), dark gray and dark grayish brown with some dark yellowish brown pockets and layers, damp, mostly nonplastic to low plasticity fines; includes some pockets/layers of SANDY LEAN CLAY (CL), low plasticity, occasional gravel layers

No desiccation cracks or other open fissures or seams observed. Core Material appears well-compacted and tight around occasional gravel layers.

EXCAVATION SKETCH / FIELD NOTES

Scale: 1 inch = 5 feet

Log of North Wall



Note: East end of trench is 10 ft west of DH-203

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CAMANCHE EMBANKMENTS SAFETY REVIEW

Log of Test Pit TP-202

Date(s) Excavated	**7/22/08**	Surface Condition	**Paved roadway**	Logged By	R. Harlan	Checked By	
Excavation Contractor	**PC Exploration**	Weather Condition	**Clear**	Length of Excavation	15 feet	Total Depth of Excavation	**8.0 feet**
Excavation Equipment	**JD-310 Backhoe**	Groundwater Observations	**Dry at time of excavation**	Width of Excavation	10 feet at surface, 2-2.5 feet in core	Approximate Surface Elevation	**266 feet MSL**
Test Pit Location	**Main Dam crest, C/L of trench 11 ft D/S of U/S edge of crest and 4 ft U/S of crest C/L, approz. Station 314+25**			Test Pit Completion	**Core zone portion backfilled with concrete; crest/slope protection gravel backfilled and compacted in approx. 1-ft lifts with vibratory compactor**		

MATERIAL DESCRIPTION

0 - 0.2 ft: Asphalt

CREST / SLOPE PROTECTION (ZONE 6)

(1) 0.2 - 4.5 ft: WELL-GRADED GRAVEL and COBBLES with SAND (GW), moderate yellowish brown, damp, some cobbles to 8 inches dia.; includes occasional silty pockets (GW-GM)

CORE (ZONE 1)

(2) 4.5 - 5.5 ft: CLAYEY GRAVEL (GC), moderate yellowish brown, moist to locally very moist, low to medium plasticity fines

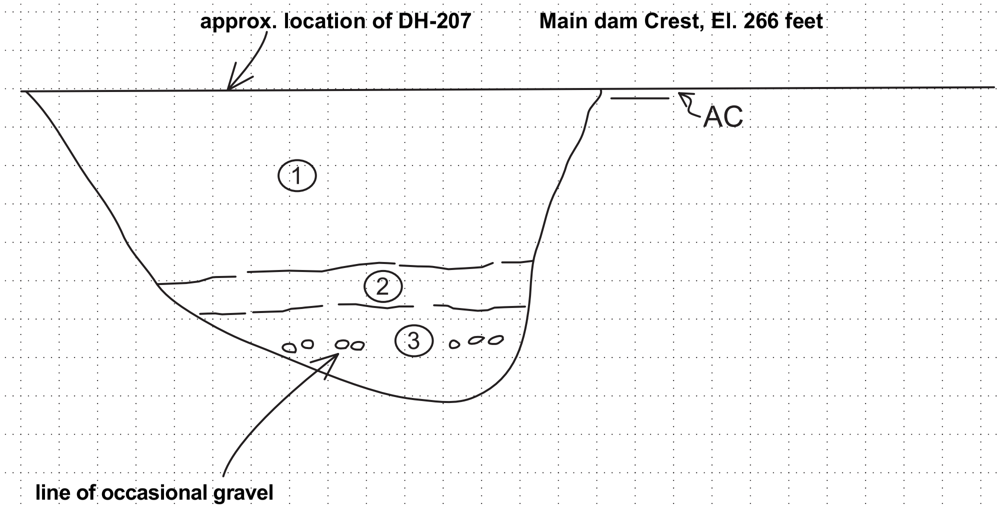
(3) 5.5 - 8 ft: SILT (ML), moderate yellowish brown, mosit, low plasticity, occasional gravel within silt core matrix; damp to moist below 7.5 ft

No desiccation cracks or toher open fissures or seams observed. Top 2 to 3 ft of core is moist to locally very moist.

EXCAVATION SKETCH / FIELD NOTES

Scale: 1 inch = 5 feet

Log of West Wall



Note: Top of core is approx. 2 feet lower on east (upstream-side) wall. Trench apparently located along top upstream edge of core. East wall of trench (mostly gravel and cobbles) readily collapses.

TERRA / GeoPentech
a Joint Venture

Report: TGP TESTPIT LAND; File: TGP_CAMANCHE.GPJ; 8/24/2008

CAMANCHE EMBANKMENTS SAFETY REVIEW

Log of Test Pit TP-203

Date(s) Excavated	**7/21/08**	Surface Condition	**Gravel roadway along dike crest**	Logged By	**R. Harlan**	Checked By	
Excavation Contractor	**PC Exploration**	Weather Condition	**Clear**	Length of Excavation	**18 feet**	Total Depth of Excavation	**7.5 feet**
Excavation Equipment	**JD-310 Backhoe**	Groundwater Observations	**Dry at time of excavation**	Width of Excavation	**10 feet at surface, 2 feet through core**	Approximate Surface Elevation	**266 feet MSL**
Test Pit Location	**Dike 5 crest, about midpoint along crest length**			Test Pit Completion	**Core zone portion backfilled with concrete; crest/slope protection gravel backfilled and compacted in approx. 1-ft lifts with vibratory compactor**		

MATERIAL DESCRIPTION

CREST / SLOPE PROTECTION (ZONE 6)

(1) 0 - 3.6 ft: WELL-GRADED GRAVEL and COBBLES with SAND (GW) to SILTY GRAVEL and COBBLES with SAND (GM), moderate to dark yellowish brown, dry to 1 ft, slightly damp to damp below 1 ft, some cobbles to 8 inches dia.

CORE (ZONE 1)

(2) 3.6 - 4.5 ft: SANDY SILT (ML), moderate brown, damp to moist, low plasticity fines, fine-grained sand

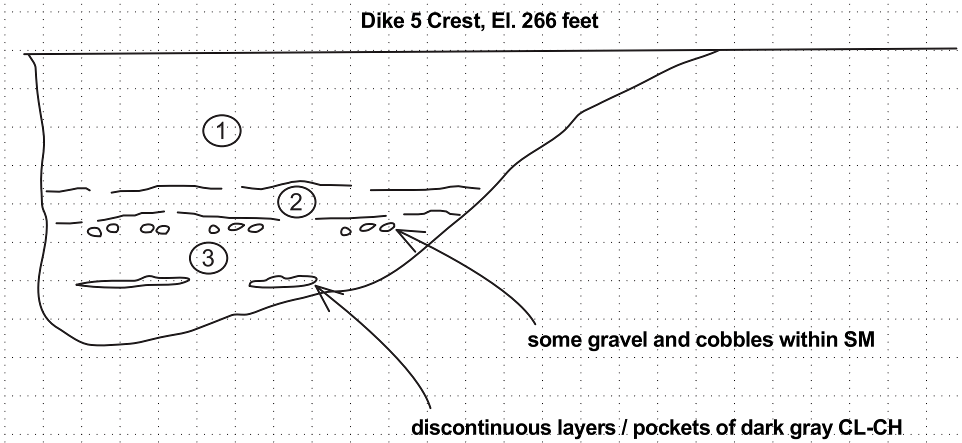
(3) 4.5 - 7.5 ft: SILTY SAND (SM), dark yellowish brown, moist, low plasticity fines, fine grained sand; includes some gravel and cobbles to 4 inches dia. (4.5-5 ft) and numerous discontinuous layers/clumps of LEAN to FAT CLAY (CL/CH), dark gray, medium to high plasticity

No desiccation cracks or other open fissures or seams observed. Core is overall damp to moist.

EXCAVATION SKETCH / FIELD NOTES

Scale: 1 inch = 5 feet

Log of East Wall



Note: North end of trench is 10 feet south of DH-210

TERRA / GeoPentech
a Joint Venture

APPENDIX B

AP Engineering & Testing, Inc.

Table B-1
Summary of
Laboratory Test Results

Client: **Terra Engineers Inc.**
Job Name: **Camanche Embankments**
Job No.:

					Index Properties											Crumb Test Results				Dispersion		
Boring Number	Embankment	Sample Number	Sample Depth	Visual Description	Moisture Content	Dry Unit Weight, pcf	Classification	Plastic Limit	Liquid Limit	Plasticity Index	Liauidity Index	Pocket Pentmtr, tsf Top/Bottom	Passing no. 200	Sieve Analysis	Hydrometer	1 minute	5 minutes	10 minutes	After 10 minutes	Crumb Test	Double Hydrometer	Pinhole Test
DH-201	Dike 1	MC-1A	5.75	Olive Gray Silty CLAY	14.6	107.4						>.4.5 / >4.5										
DH-201	Dike 1	MC-1B	6.25																			
DH-201	Dike 1	MC-2A	7.75	Brownish Grag Clayey SAND	9.5	131.6	CL	10	19	9	-0.06111	>.4.5 / >4.5	41.8	X	X	G1	G2	G3	G4	X		
DH-201	Dike 1	MC-2B	8.25																			
DH-201	Dike 1	MC-3A	9.75	Light Brown Clayey SAND	11.8	120.8						>4.5 / 2.75										
DH-201	Dike 1	MC-3B	10.25																			
DH-201	Dike 1	MC-4A	11.75	Brown Sandy CLAY	14.2	118.9	CL	17	27	10	-0.282	>.4.5 / >4.5	70.8			G1	G1	G2	G4	X		
DH-201	Dike 1	MC-4B	12.25																			
DH-201	Dike 1	MC-5A	13.75	Yellowish Brown Clayey SAND	12.0	108.9						>.4.5 / >4.5										
DH-201	Dike 1	MC-5B	14.25																			
DH-202	Dike 1	MC-1A	5.25	Light Brown Clayey SAND	12.2	99.2						3.25 / 1/00										
DH-202	Dike 1	MC-1B	5.75																			
DH-202	Dike 1	MC-2A	7.25	Dark Brown Clayey SAND	9.6	118.0						4.0 / >4.5										
DH-202	Dike 1	MC-2B	7.75																			
DH-202	Dike 1	MC-3A	9.25	Olive Brown Clayey SAND	11.1	111.6	CL-ML	14	20	6	-0.47833	>.4.5 / >4.5	58.0			G1	G2	G3	G4	X		
DH-202	Dike 1	MC-3B	9.75																			
DH-202	Dike 1	MC-4A	11.25	Olive Brown Clayey SAND	10.9	112.1						>.4.5 / >4.5										
DH-202	Dike 1	MC-4B	11.75																			
DH-202	Dike 1	MC-5	13.25	Brown Clayey SAND	8.4	128.9		NP	NP	NP	NP	>.4.5 / >4.5	89.8	X	X	G1	G1	G2	G3	X		
DH-203	Dike 2	MC-1A	5.25	Brownish Gray Clayey SAND	10.2	125.5						3.75 / >4.5										
DH-203	Dike 2	MC-1B	5.75																			
DH-203	Dike 2	MC-2A	7.25	Olive Gray Clayey SAND	10.9	122.0	CL-ML	17	21	4	-1.5225	>.4.5 / >4.5	47.3	X	X	G1	G2	G3	G4	X		ND1
DH-203	Dike 2	MC-2B	7.75																			
DH-203	Dike 2	MC-3	9.25	Brown Clayey SAND	9.3	126.4						>.4.5 / >4.5										
DH-203	Dike 2	MC-4	11.25	Brown Clayey SAND	15.8	106.4	CL-ML	18	23	5	-0.45	2.5 / >4.5	49.0			G1	G1	G2	G2	X		

AP Engineering & Testing, Inc.

Table B-1
Summary of
Laboratory Test Results

Client: **Terra Engineers Inc.**
Job Name: **Camanche Embankments**
Job No.:

Boring Number	Embankment	Sample Number	Sample Depth	Visual Description	Index Properties: Moisture Content	Dry Unit Weight, pcf	Classification	Plastic Limit	Liquid Limit	Plasticity Index	Liauidity Index	Pocket Pentmtr, tsf Top/Bottom	Passing no. 200	Sieve Analysis	Hydrometer	Crumb Test Results: 1 minute	5 minutes	10 minutes	After 10 minutes	Dispersion: Crumb Test	Double Hydrometer	Pinhole Test
DH-203	Dike 2	MC-5A	13.15	Brown Clayey SAND	8.6	132.4						>.4.5 / >4.5										
DH-203	Dike 2	MC-5B	13.55																			
DH-204	Dike 2	MC-1	5.25	Brown Clayey SAND	11.9	111.1						>.4.5 / >4.5										
DH-204	Dike 2	MC-2	7.25	Brown Sandy CLAY	22.8	98.8						>.4.5 / >4.5										
DH-204	Dike 2	MC-3A	9.75	Dark Brown Clayey SAND	13.9	113.7	CL	12	25	13	0.14846	2.25 / >4.5	56.7			G1	G2	G3	G4	X		
DH-204	Dike 2	MC-3B	10.25																			
DH-204	Dike 2	MC-4A	11.25	Olive Brown Clayey SAND	12.4	120.8						>.4.5 / >4.5		.								
DH-204	Dike 2	MC-4B	11.75																			
DH-204	Dike 2	MC-5A	13.25	Brown Clayey SAND	9.7	123.0	ML	15	18	3	-1.75667	>.4.5 / >4.5	41.2	X	X	G1	G1	G2	G3	X		
DH-204	Dike 2	MC-5B	13.75																			
DH-205	Dike 3	MC-1	5.75	Gray Clayey SAND	7.4	118.4						>.4.5 / >4.5										
DH-205	Dike 3	MC-2A	7.25	Olive Gray Clayey SAND w/ gravel	9.2	123.9	CL	13	24	11	-0.34364	>.4.5 / >4.5	41.4	X	X	G1	G2	G3	G4	X		
DH-205	Dike 3	MC-2B	7.75																			
DH-205	Dike 3	MC-3A	9.25	Grayish Brown Clayey SAND	8.8	124.2						>.4.5 / >4.5										
DH-205	Dike 3	MC-3B	9.75																			
DH-205	Dike 3	MC-4	11.25	Grayish Brown Clayey SAND	7.0	133.3		NP	NP	NP	NP	>.4.5 / >4.5	35.8			G1	G3	G4	G4	X		
DH-205	Dike 3	MC-5A	13.25	Brown Clayey SAND	9.5	126.7						3.5 / >4.5										
DH-205	Dike 3	MC-5B	13.75																			
DH-206	Dike 3	MC-1A	5.25	Dark Gray Clayey SAND	8.7	124.5						>.4.5 / >4.5										
DH-206	Dike 3	MC-1B	5.75																			
DH-206	Dike 3	MC-2A	7.75	Brown Clayey SAND	9.4	128.7						>.4.5 / >4.5										
DH-206	Dike 3	MC-2B	8.25																			
DH-206	Dike 3	MC-3A	9.75	Brown Clayey SAND	10.5	121.8	CL	12	20	8	-0.19125	>.4.5 / >4.5	54.3			G1	G2	G4	G4	X		
DH-206	Dike 3	MC-3B	10.25																			
DH-206	Dike 3	MC-4A	11.75	Dark Brown Clayey SAND	10.3	121.8						>.4.5 / >4.5										

AP Engineering & Testing, Inc.

Table B-1
Summary of
Laboratory Test Results

Client: **Terra Engineers Inc.**
Job Name: **Camanche Embankments**
Job No.:

					Index Properties											Crumb Test Results				Dispersion		
Boring Number	Embankment	Sample Number	Sample Depth	Visual Description	Moisture Content	Dry Unit Weight, pcf	Classification	Plastic Limit	Liquid Limit	Plasticity Index	Liauidity Index	Pocket Pentmtr, tsf Top/Bottom	Passing no. 200	Sieve Analysis	Hydrometer	1 minute	5 minutes	10 minutes	After 10 minutes	Crumb Test	Double Hydrometer	Pinhole Test
DH-206	Dike 3	MC-4B	12.25																			
DH-206	Dike 3	MC-5	13.25	Dark Brown Clayey SAND	11.3	119.6	CL	11	21	10	0.031	2.5 / >4.5	51.5	X	X	G1	G2	G4	G4	X		ND1
DH-207	Main Dam	MC-1	5.25	Light Brown Clayey SILT	12.9	106.9						2.5 / >4.5										
DH-207	Main Dam	MC-2	7.25	Olive Brown Clayey SILT	28.9	90.9	ML	26	38	12	0.24333	2.75 / >4.5	82.2	X	X	G1	G2	G2	G3	X		ND1
DH-207	Main Dam	MC-3	9.75	Brown Clayey SILT	22.7	101.3						3.0 / >4.5										
DH-207	Main Dam	MC-4	11.25	Grayish Brown Silty CLAY	21.6	99.5	CL	19	30	11	0.23636	2.25 / >4.5	68.3			G1	G1	G1	G2	X		
DH-207	Main Dam	MC-5A	13.25	Grayish Brown Silty CLAY	21.4	100.6						2.0 / >4.5										
DH-207	Main Dam	MC-5B	13.65																			
DH-208	Main Dam	MC-1	5.25	Light Brown Sandy SILT	18.5	100.7						2.5 / >4.5										
DH-208	Main Dam	MC-2	7.25	Light Brown Sandy SILT	24.6	97.6	CL	24	37	13	0.04846	3.25 / >4.5	95.7	X	X	G1	G1	G2	G3	X		
DH-208	Main Dam	MC-3A	11.25	Olive Gray Silty CLAY	19.1	84.3						>.4.5 / >4.5										
DH-208	Main Dam	MC-3B	11.75																			
DH-208	Main Dam	MC-4	13.25	Olive Brown SILT	21.0	109.2	CL	20	29	9	0.10667	>.4.5 / >4.5	73.9			G1	G1	G2	G2	X		
DH-208	Main Dam	MC-5A	15.25	Gray Sandy SILT	12.5	124.1						4.0 / >4.5										
DH-208	Main Dam	MC-5B	15.75																			
DH-209	Dike 4	MC-1	5.25	Dark Gray Sandy SILT	13.4	119.4						>.4.5 / >4.5										
DH-209	Dike 4	MC-2	7.75	Brown Clayey SAND	9.0	118.9						>.4.5 / >4.5										
DH-209	Dike 4	MC-3	9.75	Brown Clayey SAND	16.6	112.5		NP	NP	NP	NP	1.5 / 3.25	44.2			G1	G2	G3	G4	X		
DH-209	Dike 4	MC-4A	11.25	Brown Clayey SILT	17.4	107.2						>.4.5 / >4.5										
DH-209	Dike 4	MC-4B	11.75																			
DH-209	Dike 4	MC-5A	13.25	Brown Clayey SILT	13.5	113.6	CL-ML	16	22	6	-0.42	>.4.5 / >4.5	67.3	X	X	G1	G2	G3	G4	X		
DH-209	Dike 4	MC-5B	13.75																			

AP Engineering & Testing, Inc.

Table B-1
Summary of
Laboratory Test Results

Client: **Terra Engineers Inc.**
Job Name: **Camanche Embankments**
Job No.:

					Index Properties											Crumb Test Results				Dispersion		
Boring Number	Embankment	Sample Number	Sample Depth	Visual Description	Moisture Content	Dry Unit Weight, pcf	Classification	Plastic Limit	Liquid Limit	Plasticity Index	Liauidity Index	Pocket Pentmtr, tsf Top/Bottom	Passing no. 200	Sieve Analysis	Hydrometer	1 minute	5 minutes	10 minutes	After 10 minutes	Crumb Test	Double Hydrometer	Pinhole Test
DH-210	Dike 5	MC-1	5.75	Light Brown Clayey SAND	14.5	112.5		NP	NP	NP	NP	>.4.5 / >4.5	38.9	X	X	G1	G2	G2	G3	X		
DH-210	Dike 5	MC-2A	7.25	Light Brown Clayey SAND	13.8	116.6						>.4.5 / >4.5										
DH-210	Dike 5	MC-2B	7.75																			
DH-210	Dike 5	MC-3	9.75	Light Brown Clayey SAND	19.5	104.7						>.4.5 / >4.5										
DH-210	Dike 5	MC-4A	11.25	Light Brown Sandy CLAY	15.1	110.1	CL	15	27	12	0.00583	>.4.5 / >4.5	57.7			G1	G3	G4	G4	X		
DH-210	Dike 5	MC-4B	11.75																			
DH-210	Dike 5	MC-5A	13.25	Grayish Brown Sandy CLAY	18.9	109.4						>.4.5 / 3.75										
DH-210	Dike 5	MC-5B	13.75																			
DH-211	Dike 6	MC-1A	5.25	Grayish Brown Sandy CLAY	11.9	110.5						>.4.5 / >4.5										
DH-211	Dike 6	MC-1B	5.75																			
DH-211	Dike 6	MC-2	7.15	Grayish Brown Sandy CLAY	11.2	125.3						>.4.5 / >4.5										
DH-211	Dike 6	MC-3A	9.25	Grayish Brown Sandy CLAY	11.0	124.2	CL	12	21	9	-0.10667	2.25 / >4.5	53.6			G1	G3	G4	G4	X		ND1
DH-211	Dike 6	MC-3B	9.75																			
DH-211	Dike 6	MC-4A	11.75	Grayish Brown Sandy CLAY	12.5	119.0						>.4.5 / >4.5										
DH-211	Dike 6	MC-4B	12.25																			
DH-211	Dike 6	MC-5A	13.75	Grayish Brown Sandy CLAY	12.7	121.1	CL-ML	15	20	5	-0.452	2.75 / 4.25	43.8	X	X	G1	G3	G4	G4	X		
DH-211	Dike 6	MC-5B	14.25																			

Special Instruction: Provide pocket penetrometer and visual classification on all samples tested for water content and unit weight

MOISTURE AND DENSITY TEST RESULTS

Client: Terra Engineers Inc.
Project Name: Camanche Embankments
Project No: NA

AP No.: 28-0755
Date: 07/29/08

Boring No.	Sample No.	Sample Depth (ft)	Visual Soil Description	Moisture Content (%)	Dry Density (pcf)
DH-201	MC-1A	5.75	Olive Gray Silty Clay	14.58	107.42
DH-201	MC-2A	7.75	Brownish Gray Clayey Sand	9.45	131.64
DH-201	MC-3A	9.75	Light Brown Clayey Sand	11.82	120.80
DH-201	MC-4A	11.75	Brown Sandy Clay	14.18	118.93
DH-201	MC-5A	13.75	Yellowish Brown Clayey Sand	12.01	108.94

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: Terra Engineers Inc.
Project Name: Camanche Embankments
Project No: NA

AP No.: 28-0755
Date: 07/29/08

Boring No.	Sample No.	Sample Depth (ft)	Visual Soil Description	Moisture Content (%)	Dry Density (pcf)
DH-202	MC-1A	2.25	Light Brown Clayey Sand	12.16	99.17
DH-202	MC-2A	7.25	Dark Brown Clayey Sand	9.62	118.02
DH-202	MC-3A	9.25	Olive Brown Clayey Sand	11.13	111.55
DH-202	MC-4A	11.25	Olive Brown Clayey Sand	10.92	112.13
DH-202	MC-5	13.25	Brown Clayey Sand	8.37	128.88

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: Terra Engineers Inc.
Project Name: Camanche Embankments
Project No: NA

AP No.: 28-0755
Date: 07/29/08

Boring No.	Sample No.	Sample Depth (ft)	Visual Soil Description	Moisture Content (%)	Dry Density (pcf)
DH-203	MC-1A	5.25	Brownish Gray Clayey Sand	10.20	125.52
DH-203	MC-2A	7.25	Olive Gray Clayey Sand	10.91	121.97
DH-203	MC-3	9.25	Brown Clayey Sand	9.25	126.39
DH-203	MC-4	11.25	Brown Clayey Sand	15.75	106.39
DH-203	MC-5A	13.15	Brown Clayey Sand	8.60	132.36

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: Terra Engineers Inc.
Project Name: Camanche Embankments
Project No: NA

AP No.: 28-0755
Date: 07/29/08

Boring No.	Sample No.	Sample Depth (ft)	Visual Soil Description	Moisture Content (%)	Dry Density (pcf)
DH-204	MC-1	5.25	Brown Clayey Sand	11.92	111.10
DH-204	MC-2	7.25	Brown Sandy Clay	22.84	98.78
DH-204	MC-3A	9.75	Dark Brown Clayey Sand	13.93	113.69
DH-204	MC-4A	11.25	Olive Brown Clayey Sand	12.39	120.81
DH-204	MC-5A	13.25	Brown Clayey Sand	9.73	123.03

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: Terra Engineers Inc. | AP No.: 28-0755

Project Name: Camanche Embankments | Date: 07/29/08

Project No: NA

Boring No.	Sample No.	Sample Depth (ft)	Visual Soil Description	Moisture Content (%)	Dry Density (pcf)
DH-205	MC-1	5.75	Gray Clayey Sand	7.43	118.41
DH-205	MC-2A	7.25	Olive Gray Clayey Sand w/gravel	9.22	123.94
DH-205	MC-3A	9.25	Grayish Brown Clayey Sand	8.79	124.16
DH-205	MC-4	11.25	Grayish Brown Clayey Sand	6.95	133.25
DH-205	MC-5A	13.75	Brown Clayey Sand	9.50	126.69

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: Terra Engineers Inc. AP No.: 28-0755

Project Name: Camanche Embankments Date: 07/29/08

Project No: NA

Boring No.	Sample No.	Sample Depth (ft)	Visual Soil Description	Moisture Content (%)	Dry Density (pcf)
DH-206	MC-1A	5.25	Dark Gray Clayey Sand	8.74	124.50
DH-206	MC-2A	7.75	Brown Clayey Sand	9.40	128.73
DH-206	MC-3A	9.75	Brown Clayey Sand	10.47	121.82
DH-206	MC-4A	11.75	Dark Brown Clayey Sand	10.34	121.79
DH-206	MC-5	13.25	Dark Brown Clayey Sand	11.31	119.57

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: Terra Engineers Inc. AP No.: 28-0755

Project Name: Camanche Embankments Date: 07/29/08

Project No: NA

Boring No.	Sample No.	Sample Depth (ft)	Visual Soil Description	Moisture Content (%)	Dry Density (pcf)
DH-207	MC-1	5.25	Light Brown Clayey Silt	12.94	106.93
DH-207	MC-2	7.25	Olive Brown Clayey Silt	28.92	90.94
DH-207	MC-3	9.75	Brown Clayey Silt	22.74	101.31
DH-207	MC-4	11.25	Grayish Brown Silty Clay	21.60	99.54
DH-207	MC-5A	13.25	Grayish Brown Silty Clay	21.42	100.62

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: Terra Engineers Inc.
Project Name: Camanche Embankments
Project No: NA

AP No.: 28-0755
Date: 07/29/08

Boring No.	Sample No.	Sample Depth (ft)	Visual Soil Description	Moisture Content (%)	Dry Density (pcf)
DH-208	MC-1	5.25	Light Brown Sandy Silt	18.45	100.68
DH-208	MC-2	7.25	Light Brown Sandy Silt	24.63	97.64
DH-208	MC-3A	11.25	Olive Gray Silty Clay	19.05	84.27
DH-208	MC-4	13.25	Olive Brown Silt	20.96	109.19
DH-208	MC-5A	15.25	Gray Sandy Silt	12.53	124.06

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: Terra Engineers Inc.
Project Name: Camanche Embankments
Project No: NA

AP No.: 28-0755
Date: 07/29/08

Boring No.	Sample No.	Sample Depth (ft)	Visual Soil Description	Moisture Content (%)	Dry Density (pcf)
DH-209	MC-1	5.25	Dark Gray Sandy Silt	13.39	119.40
DH-209	MC-2	7.75	Brown Clayey Sand	9.01	118.92
DH-209	MC-3	9.75	Brown Clayey Sand	16.57	112.47
DH-209	MC-4A	11.25	Brown Clayey Silt	17.39	107.20
DH-209	MC-5A	13.25	Brown Clayey Silt	13.48	113.58

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: Terra Engineers Inc. AP No.: 28-0755

Project Name: Camanche Embankments Date: 07/29/08

Project No: NA

Boring No.	Sample No.	Sample Depth (ft)	Visual Soil Description	Moisture Content (%)	Dry Density (pcf)
DH-210	MC-1	5.75	Light Brown Clayey Sand	14.52	112.50
DH-210	MC-2A	7.25	Light Brown Clayey Sand	13.78	116.55
DH-210	MC-3	9.75	Light Brown Clayey Sand	19.53	104.71
DH-210	MC-4A	11.25	Light Brown Sandy Clay	15.07	110.06
DH-210	MC-5A	13.25	Grayish Brown Sandy Clay	18.89	109.42

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: Terra Engineers Inc.
AP No.: 28-0755
Project Name: Camanche Embankments
Date: 07/29/08
Project No: NA

Boring No.	Sample No.	Sample Depth (ft)	Visual Soil Description	Moisture Content (%)	Dry Density (pcf)
DH-211	MC-1A	5.25	Grayish Brown Sandy Clay	11.90	110.45
DH-211	MC-2	7.15	Grayish Brown Sandy Clay	11.15	125.32
DH-211	MC-3A	9.25	Grayish Brown Sandy Clay	11.04	124.19
DH-211	MC-4A	11.75	Grayish Brown Sandy Clay	12.47	118.96
DH-211	MC-5A	13.75	Grayish Brown Sandy Clay	12.74	121.06

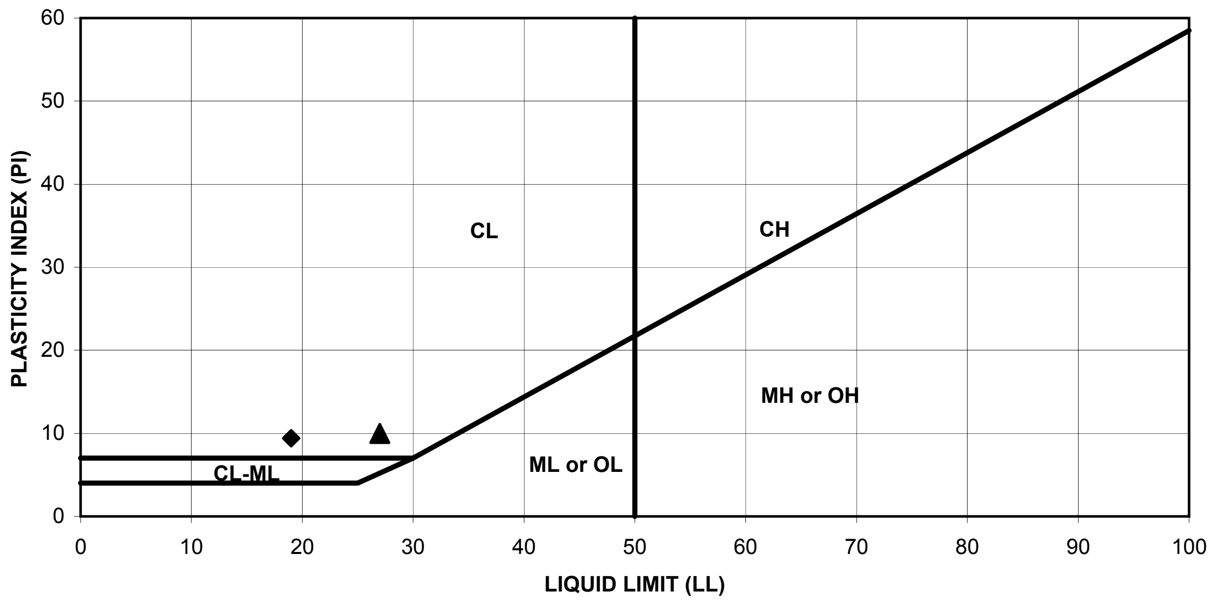
AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

Project Name: Camanche Embankments

Tested By: DK Date: 07/30/08

Checked By: AP Date: 07/31/08



PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test

Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
♦	DH-201	MC-2A	7.75	19	10	9	CL
▲	DH-201	MC-4A	11.75	27	17	10	CL

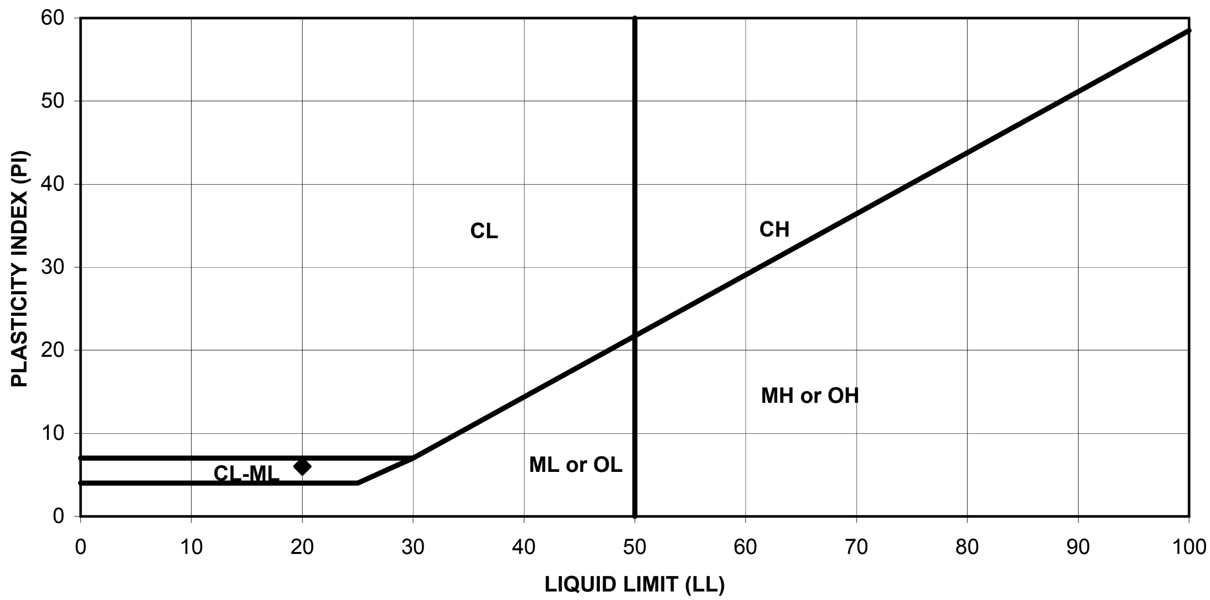
AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

Project Name: Camanche Embankments

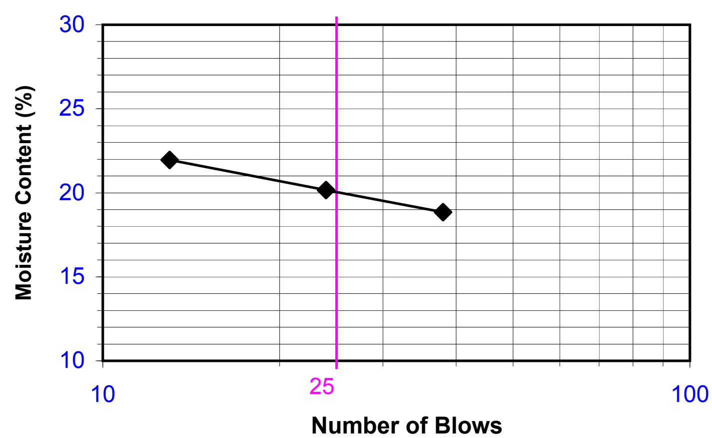
Tested By: DK Date: 07/30/08

Checked By: AP Date: 07/31/08



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
♦	DH-202	MC-3A	9.25	20	14	6	CL-ML
	DH-202	MC-5	13.25	NP	NP	NP	

* NP denotes "non-plastic"

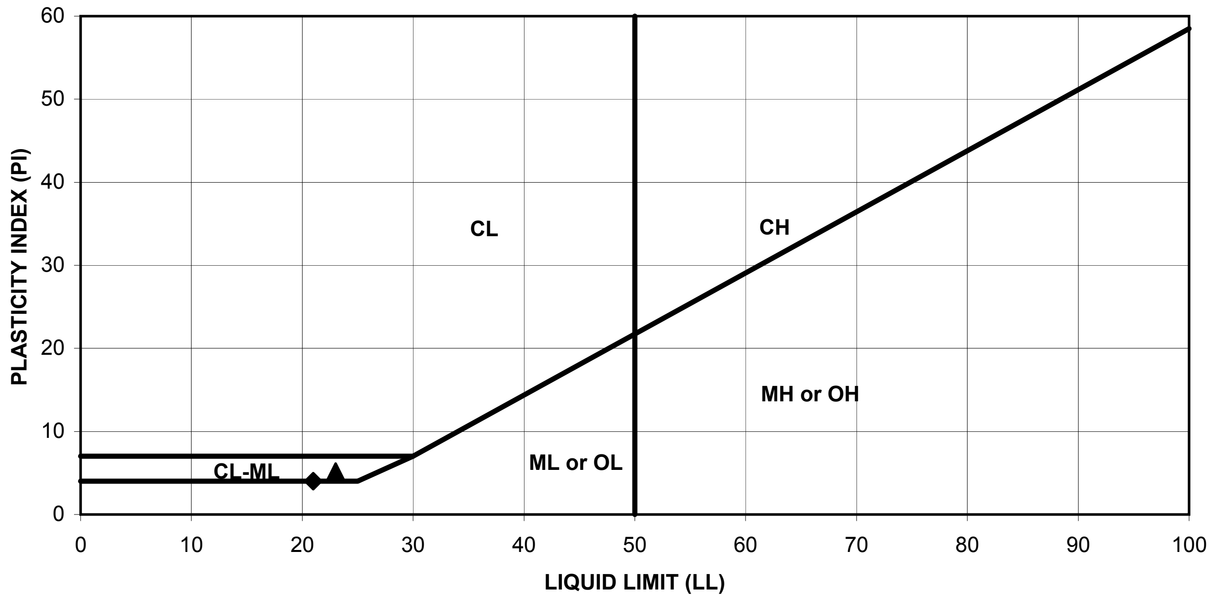
AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

Project Name: Camanche Embankments

Tested By: DK — Date: 07/30/08

Checked By: AP — Date: 07/31/08



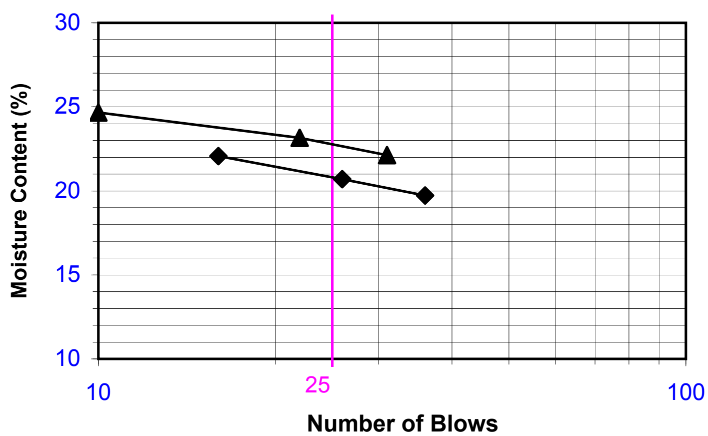
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



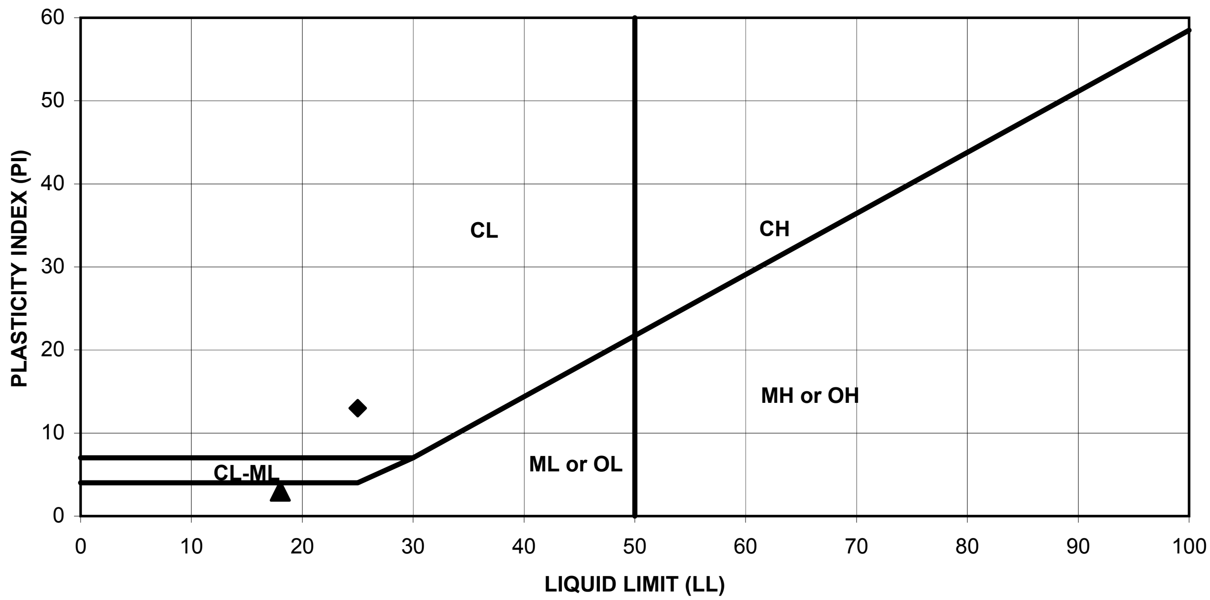
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
◆	DH-203	MC-2A	7.25	21	17	4	CL-ML
▲	DH-203	MC-4	11.25	23	18	5	CL-ML

AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

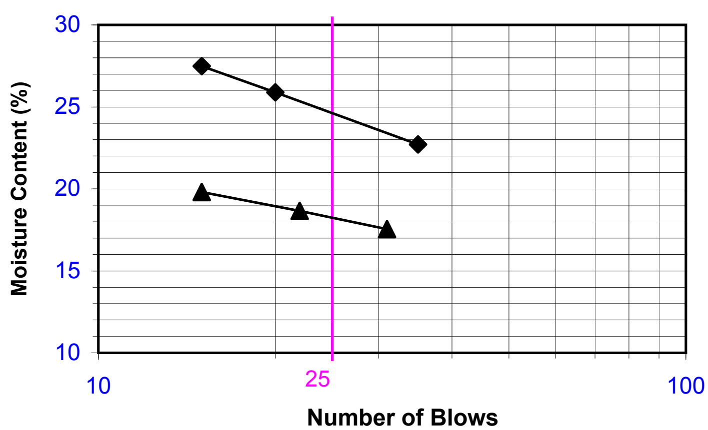
Project Name: Camanche Embankments

Tested By: DK   Date: 07/30/08
Checked By: AP   Date: 07/31/08



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test

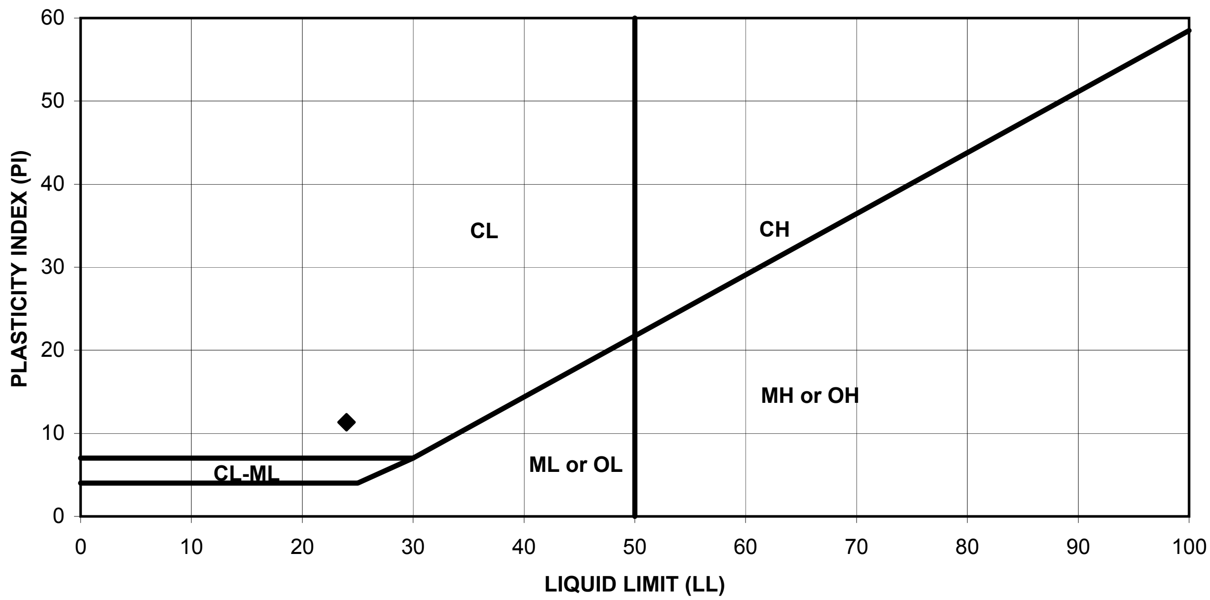


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
◆	DH-204	MC-3A	9.75	25	12	13	CL
▲	DH-204	MC-5A	13.25	18	15	3	ML

AP Engineering & Testing, Inc.

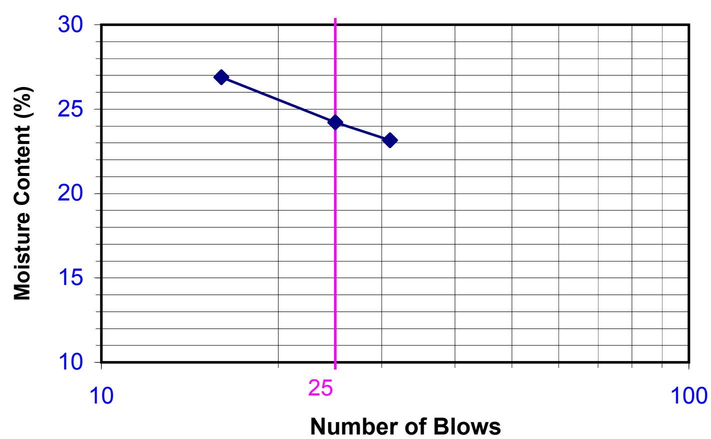
ATTERBERG LIMITS
ASTM D 4318

Project Name:	Camanche Embankments	Tested By:	DK	Date:	07/30/08
		Checked By:	AP	Date:	08/04/08



PROCEDURE USED

- [ ] Wet Preparation
- [x] Dry Preparation
- [x] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
♦	DH-205	MC-2A	7.25	24	13	11	CL
	DH-205	MC-4	11.25	NP	NP	NP	

* NP denotes "non-plastic"

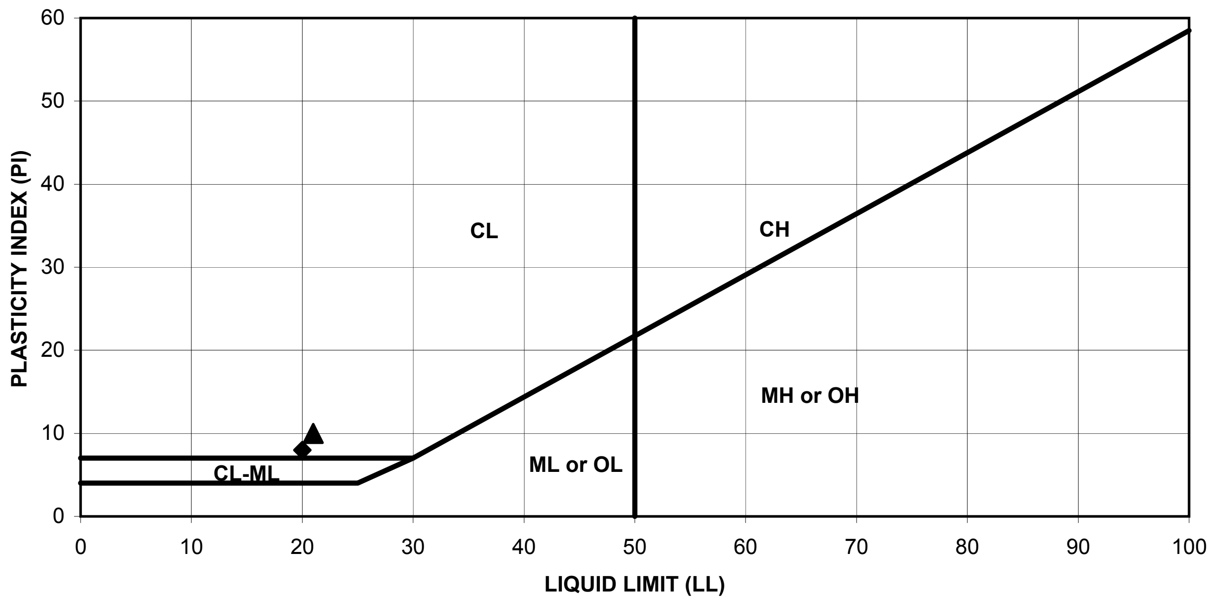
AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

Project Name: Camanche Embankments

Tested By: DK   Date: 07/30/08

Checked By: AP   Date: 08/04/08



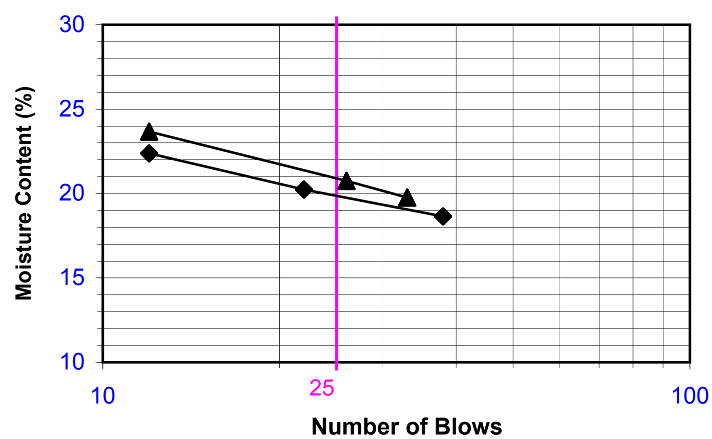
PROCEDURE USED

[ ] Wet Preparation

[X] Dry Preparation

[X] Procedure A
Multipoint Test

[ ] Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
♦	DH-206	MC-3A	9.75	20	12	8	CL
▲	DH-206	MC-5	13.25	21	11	10	CL

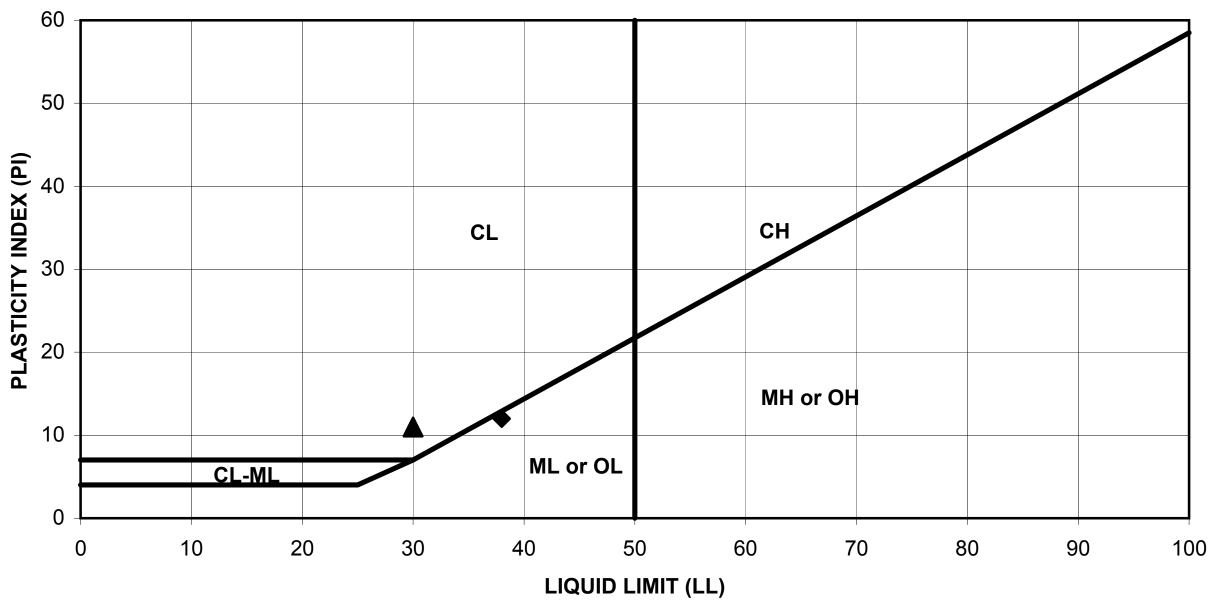
AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

Project Name: Camanche Embankments

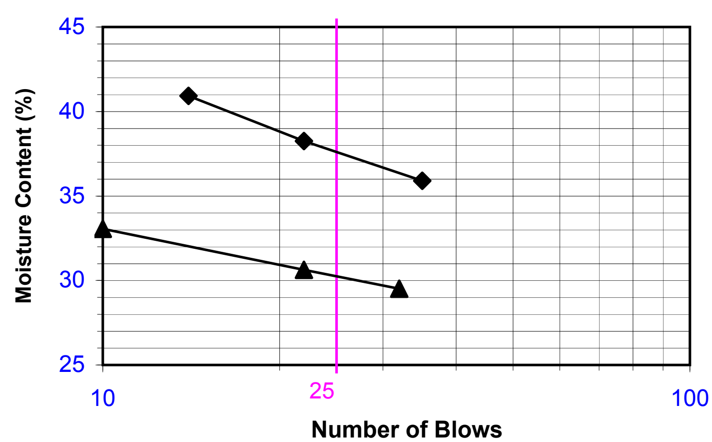
Tested By: DK Date: 07/30/08

Checked By: AP Date: 08/04/08



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
♦	DH-207	MC-2	7.25	38	26	12	ML
▲	DH-207	MC-4	11.25	30	19	11	CL

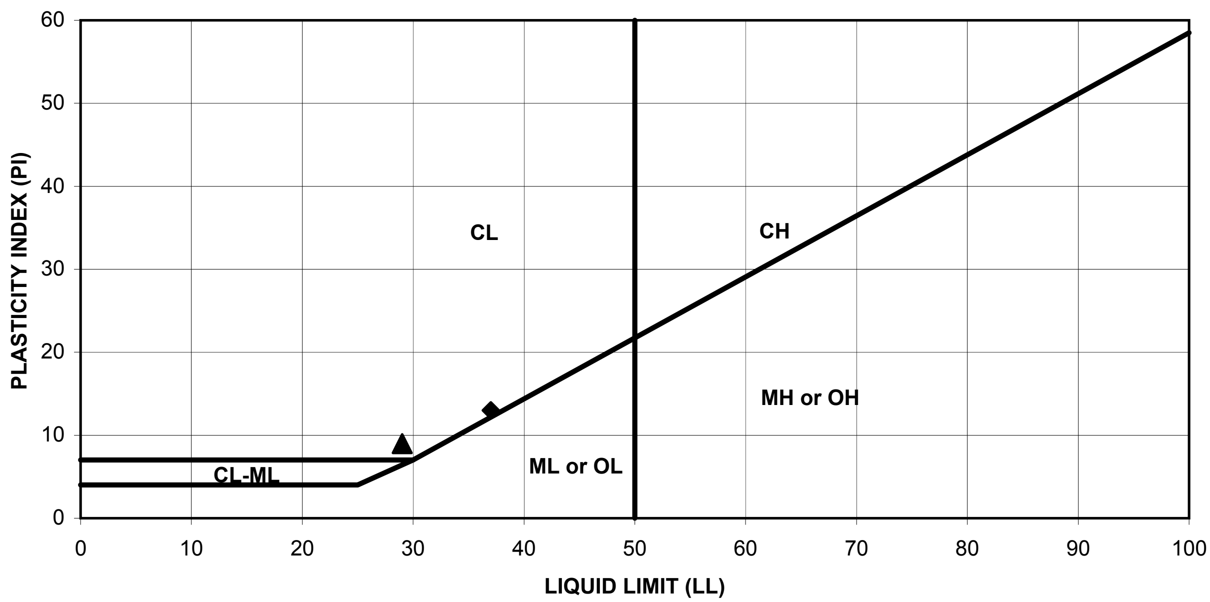
AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

Project Name: Camanche Embankments

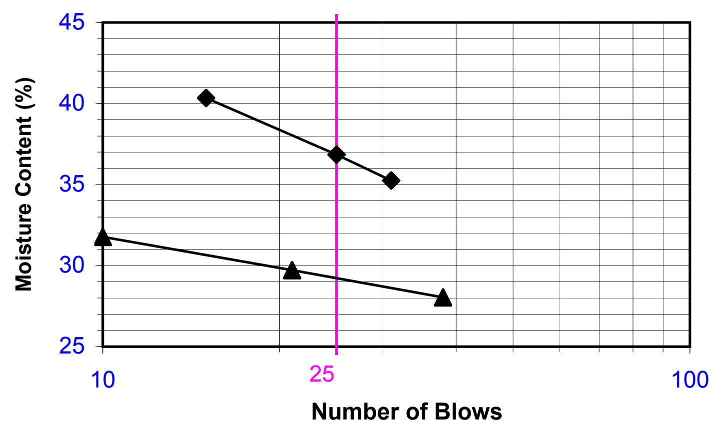
Tested By: DK Date: 07/30/08

Checked By: AP Date: 08/04/08



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
♦	DH-208	MC-2	7.25	37	24	13	CL
▲	DH-208	MC-4	13.25	29	20	9	CL

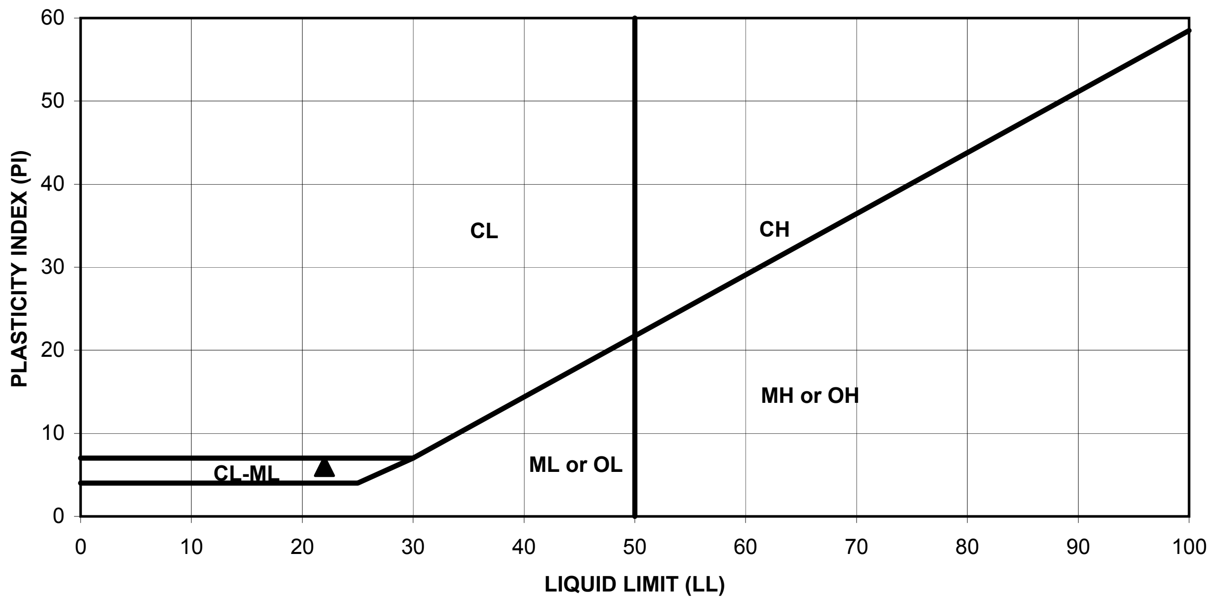
AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

Project Name: Camanche Embankments

Tested By: DK — Date: 07/30/08

Checked By: AP — Date: 08/06/08



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test

Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
	DH-209	MC-3	9.75	NP	NP	NP	
▲	DH-209	MC-5A	13.25	22	16	6	CL-ML

* NP denotes "non-plastic"

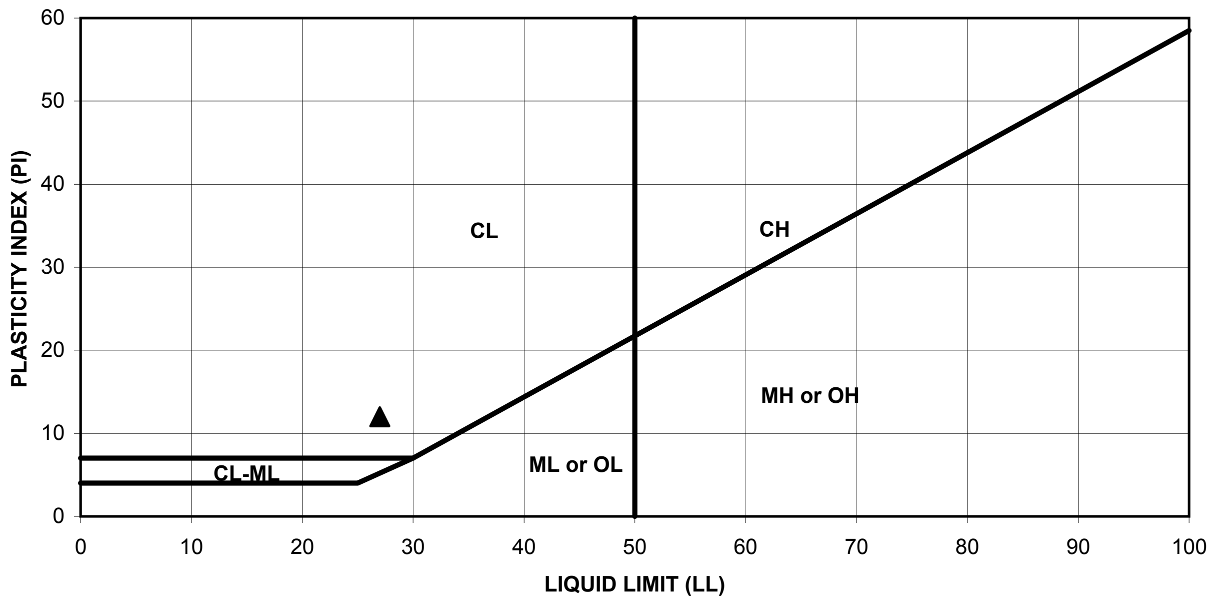
AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

Project Name: Camanche Embankments

Tested By: DK   Date: 07/30/08

Checked By: AP   Date: 08/06/08



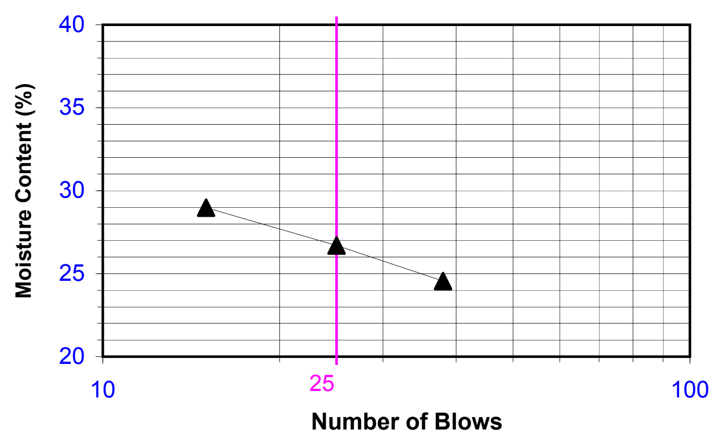
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
	DH-210	MC-1	5.75	NP	NP	NP	
▲	DH-210	MC-4A	11.25	27	15	12	

* NP denotes "non-plastic"

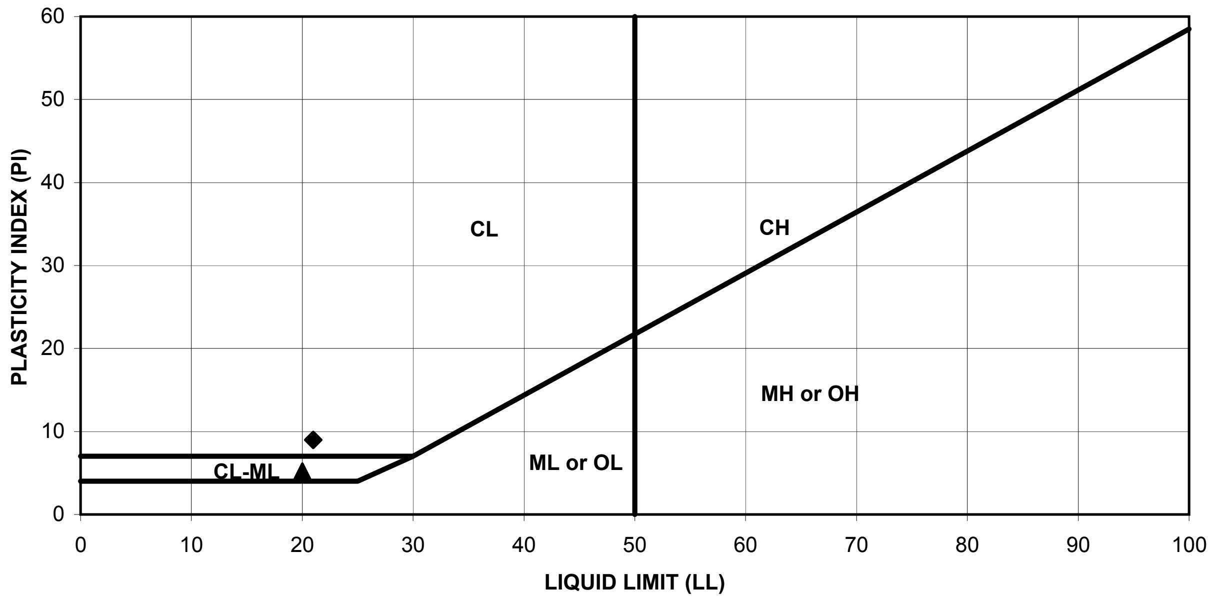
AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

Project Name: Camanche Embankments

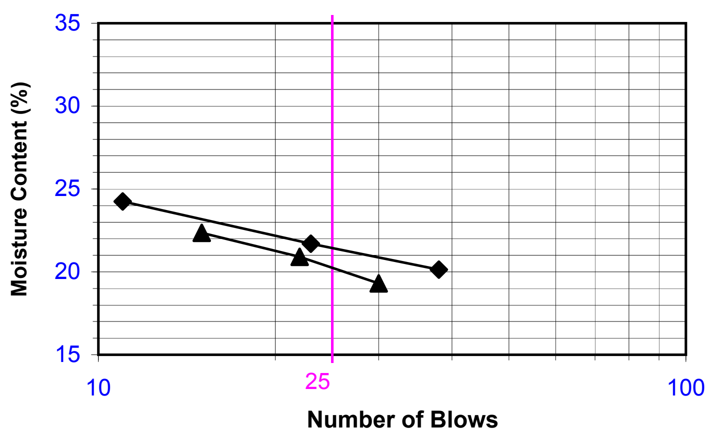
Tested By: DK Date: 07/30/08

Checked By: AP Date: 08/06/08



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	U.S.C.S Symbol
◆	DH-211	MC-3A	9.25	21	12	9	CL
▲	DH-211	MC-5A	13.75	20	15	5	CL-ML

AP Engineering & Testing, Inc.

PERCENT PASSING NO. 200 SIEVE

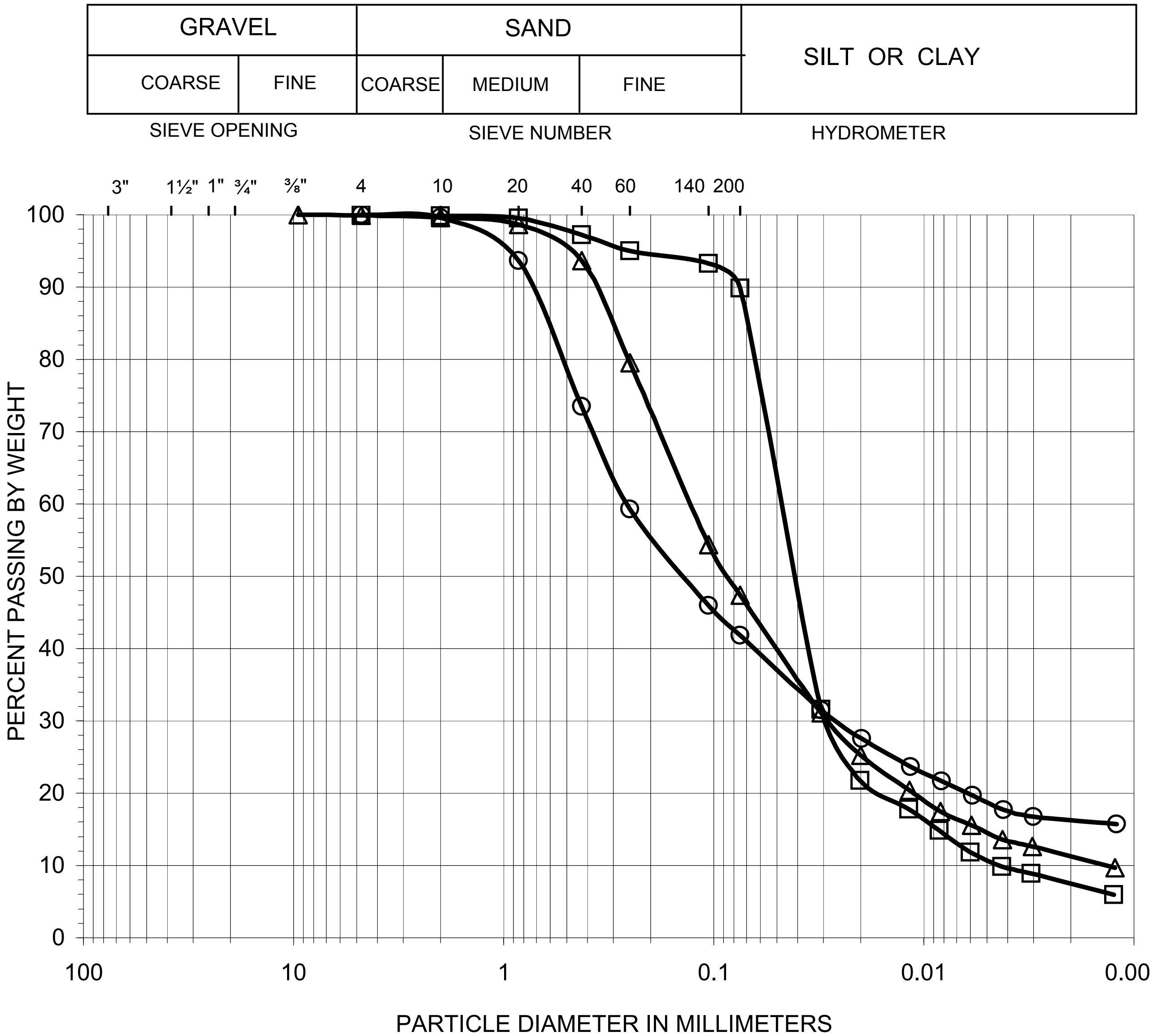
Client: Terra Engineers Inc. Laboratory No.: 28-0755

Project Name: Camanche Embankments Date: 07/28/08

Project Location: NA

Boring No.	Sample No.	Sample Depth (ft)	Soil Description	Percent Fines (%)
DH-201	MC-2A	7.75	Brownish Gray Clayey Sand	41.8
DH-201	MC-4A	11.75	Brown Sandy Clay	70.8
DH-202	MC-3A	9.25	Olive Brown Sandy Silt-Sandy Clay	58.0
DH-202	MC-5	13.25	Olive Brown Silt with fine sand	89.8
DH-203	MC-2A	7.25	Olive Gray Silty-Clayey Sand	47.3
DH-203	MC-4	11.25	Brown Clayey Sand	49.0
DH-204	MC-3A	9.75	Dark Brown Sandy Clay	56.7
DH-204	MC-5A	13.25	Brown Silty Sand	41.2
DH-205	MC-2A	7.25	Olive Gray Clayey Sand	41.4
DH-205	MC-4	11.25	Grayish Brown Clayey Sand	35.8
DH-206	MC-3A	9.75	Brown Sandy Clay	54.3
DH-206	MC-5	13.25	Dark Brown Sandy Clay	51.5
DH-207	MC-2	7.25	Olive Brown Lean Clay	82.2
DH-207	MC-4	11.25	Grayish Brown Silty Clay	68.3
DH-208	MC-2	7.25	Light Brown Lean Clay	95.7
DH-208	MC-4	13.25	Olive Brown Silt	73.9
DH-209	MC-3	9.75	Brown Clayey Sand	44.2
DH-209	MC-5A	13.25	Brown Silty Clay with sand	67.3
DH-210	MC-1	5.75	Light Brown Silty Sand	38.9
DH-210	MC-4A	11.25	Light Brown Sandy Clay	57.7
DH-211	MC-3A	9.25	Grayish Brown Sandy Clay	53.6
DH-211	MC-5A	13.75	Grayish Brown Clayey Sand	43.8

2607 Pomona Boulevard. Pomona, California 91768
Tel. (909) 869-6316, Fax (909) 869-6318



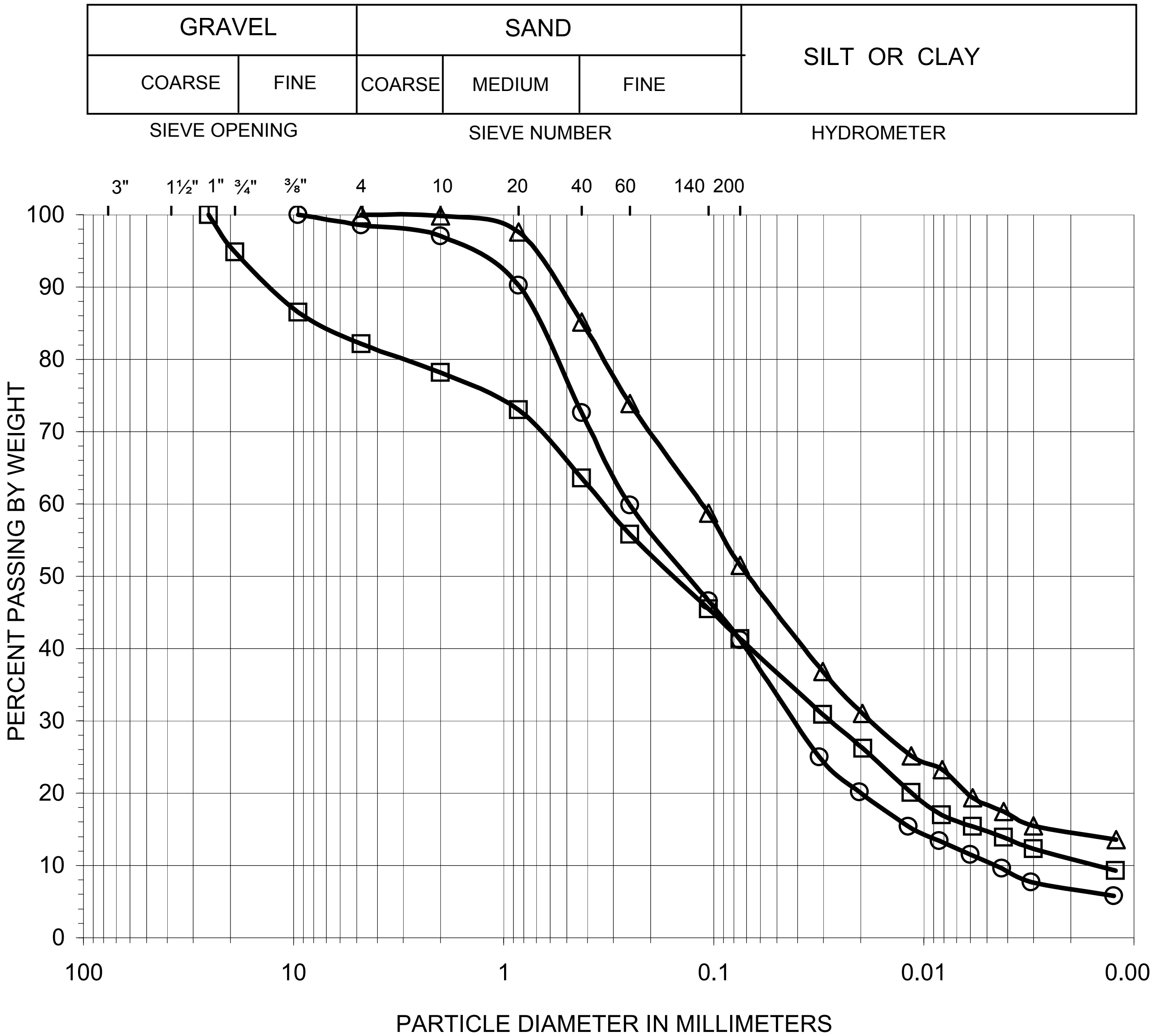
Symbol	Boring No.	Sample No.	Depth (ft)	Percent			Atterberg Limits LL:PL:PI	U.S.C.S Symbol
				Gravel	Sand	Fines		
○	DH-201	MC-2A	7.75	0.0	58.2	41.8	19:10:9	SC
□	DH-202	MC-5	13.25	0.0	10.2	89.8	NP	ML
△	DH-203	MC-2A	7.25	0.1	52.6	47.3	21:17:4	SM-SC

NP = Non-plastic

GRAIN SIZE DISTRIBUTION CURVE

ASTM D 422

Project Name: Camanche Embankments
Project No.: NA
Date: 7/29/2008
AP No: 28-0755

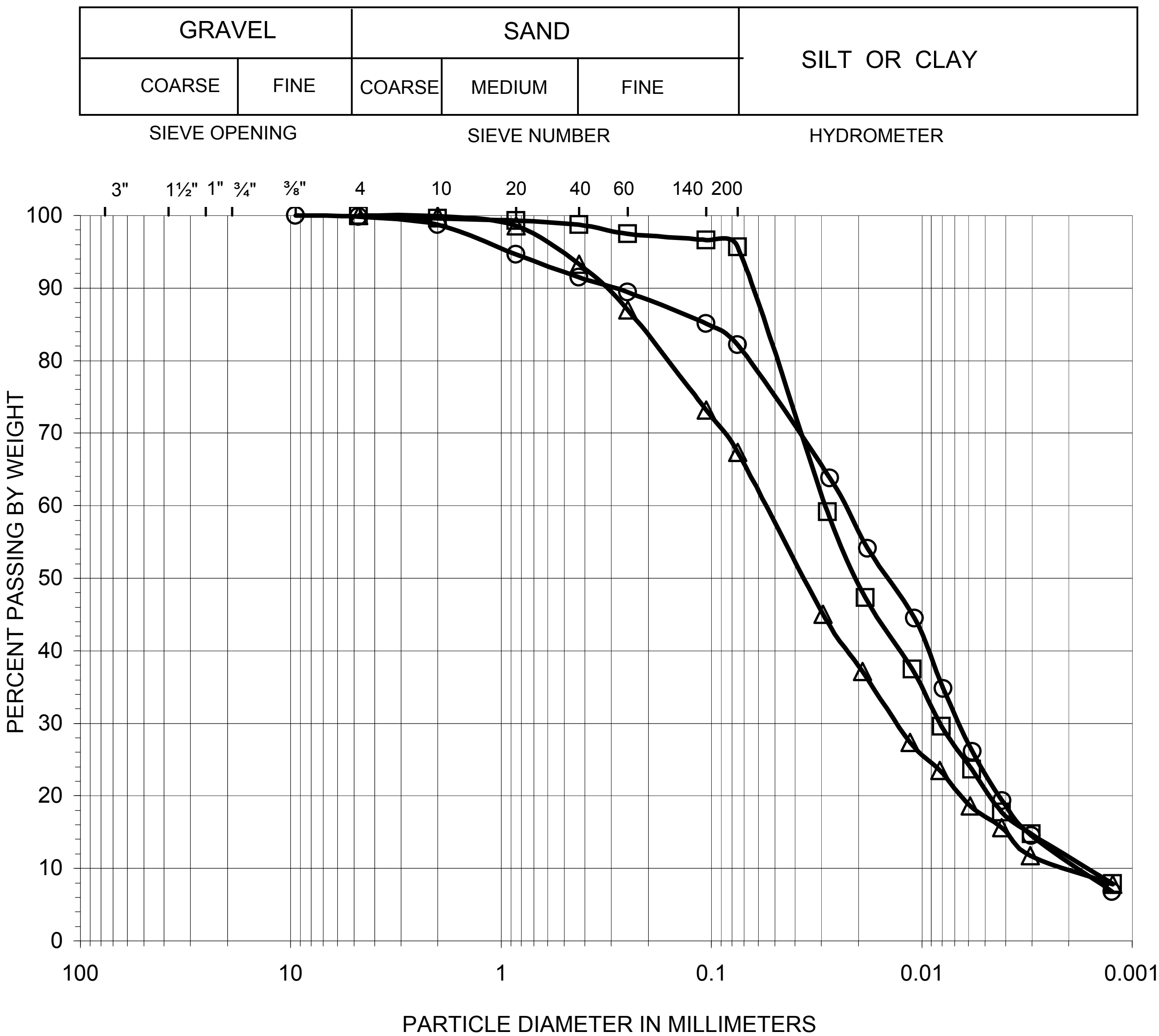


Symbol	Boring No.	Sample No.	Depth (ft)	Percent			Atterberg Limits LL:PL:PI	U.S.C.S Symbol
				Gravel	Sand	Fines		
○	DH-204	MC-5A	13.25	1.5	57.3	41.2	18:15:3	SM
□	DH-205	MC-2A	7.25	17.9	40.8	41.4	24:13:11	SC
△	DH-206	MC-5	13.25	0.0	48.5	51.5	21:11:10	CL

GRAIN SIZE DISTRIBUTION CURVE

ASTM D 422

Project Name: Camanche Embankments
Project No.: NA
Date: 7/29/2008
AP No: 28-0755



Symbol	Boring No.	Sample No.	Depth (ft)	Percent			Atterberg Limits LL:PL:PI	U.S.C.S Symbol
				Gravel	Sand	Fines		
○	DH-207	MC-2	7.25	0.2	17.6	82.2	38:26:12	CL
□	DH-208	MC-2	7.25	0.0	4.3	95.7	37:24:13	CL
△	DH-209	MC-5A	13.25	0.0	32.7	67.3	22:16:6	CL-ML

GRAIN SIZE DISTRIBUTION CURVE

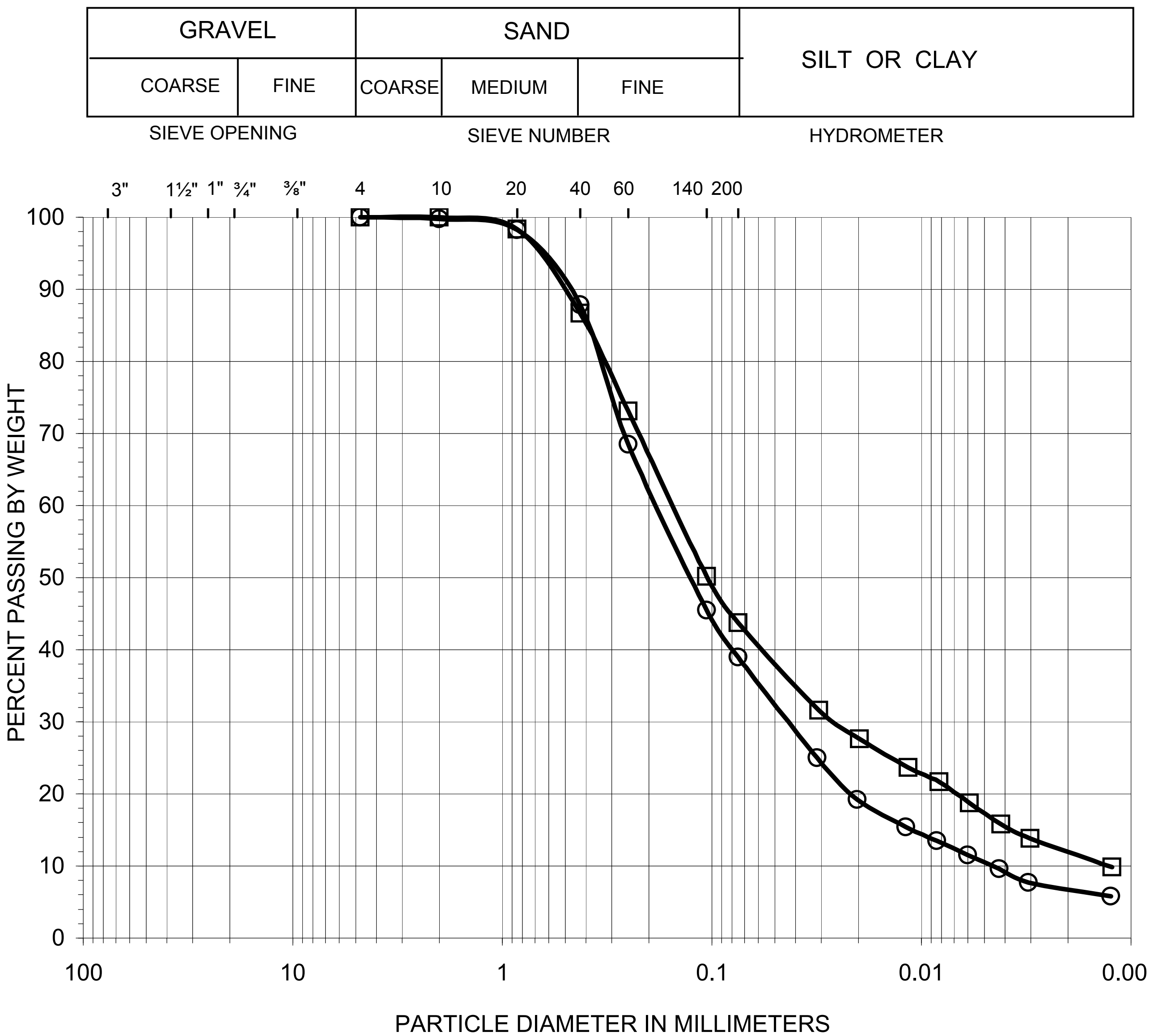
ASTM D 422

Project Name: Camanche Embankments

Project No.: NA

Date: 7/29/2008

AP No: 28-0755



Symbol	Boring No.	Sample No.	Depth (ft)	Percent			Atterberg Limits LL:PL:PI	U.S.C.S Symbol
				Gravel	Sand	Fines		
○	DH-210	MC-1	5.75	0.0	61.1	38.9	NP	SM
□	DH-211	MC-5A	13.75	0.0	56.2	43.8	21:12:9	SC

NP=Non-plastic

GRAIN SIZE DISTRIBUTION CURVE

ASTM D 422

Project Name: Camanche Embankments
Project No.: NA
Date: 7/29/2008
AP No: 28-0755

AP Engineering & Testing, Inc.

Geotechnical Testing Laboratory

CRUMB TEST RESULTS

Client: Terra Engineers, Inc.

Project Name: Camanche Embankments

Project No.: N/A

AP Job No.: 28-0755

Report Date: 08/01/08

Test Date	Boring No.	Sample No.	Depth (feet)	Results (G)			
				Time=1 min	Time = 5 min	Time = 10 min	After 10 min
7/31/2008	DH-201	MC-2A	7.75				
				G1	G2	G3	G4
7/31/2008	DH-201	MC-4A	11.75				
				G1	G1	G2	G4

Grade	Description	Reaction	Categories
G1	No Reaction	Crumbs may slake or run out to form a shallow heap on the bottom of the beaker, but there is no sign of cloudiness caused by colloidal in suspension.	Non-Dispersive
G2	Slightly Reaction	A very slightly cloudiness can be seen in the water at the surface of the crumb.	Non-Dispersive
G3	Moderate Reaction	There is easily recognizable cloud of colloidal in suspension, usually spreading out in thin streaks at the bottom of the beaker.	Dispersive
G4	Strong Reaction	A colloidal cloud covers most of the bottom of the beaker, usually as a thin skin. In extreme case, all the water becomes cloudy.	Dispersive

AP Engineering & Testing, Inc.

Geotechnical Testing Laboratory

CRUMB TEST RESULTS

Client: Terra Engineers, Inc.

Project Name: Camanche Embankments

Project No.: N/A

AP Job No.: 28-0755

Report Date: 08/01/08

Test Date	Boring No.	Sample No.	Depth (feet)	Results (G)			
				Time=1 min	Time = 5 min	Time = 10 min	After 10 min
7/31/2008	DH-202	MC-3A	9.25				
				G1	G2	G3	G4
7/31/2008	DH-202	MC-5	13.25				
				G1	G1	G2	G3

Grade	Description	Reaction	Categories
G1	No Reaction	Crumbs may slake or run out to form a shallow heap on the bottom of the beaker, but there is no sign of cloudiness caused by colloidal in suspension.	Non-Dispersive
G2	Slightly Reaction	A very slightly cloudiness can be seen in the water at the surface of the crumb.	Non-Dispersive
G3	Moderate Reaction	There is easily recognizable cloud of colloidal in suspension, usually spreading out in thin streaks at the bottom of the beaker.	Dispersive
G4	Strong Reaction	A colloidal cloud covers most of the bottom of the beaker, usually as a thin skin. In extreme case, all the water becomes cloudy.	Dispersive

AP Engineering & Testing, Inc.

Geotechnical Testing Laboratory

CRUMB TEST RESULTS

Client: Terra Engineers, Inc.

Project Name: Camanche Embankments

Project No.: N/A

AP Job No.: 28-0755

Report Date: 08/01/08

Test Date	Boring No.	Sample No.	Depth (feet)	Results (G)			
				Time=1 min	Time = 5 min	Time = 10 min	After 10 min
7/31/2008	DH-203	MC-2A	7.25				
				G1	G2	G3	G4
7/31/2008	DH-203	MC-4	11.25				
				G1	G1	G2	G2

Grade	Description	Reaction	Categories
G1	No Reaction	Crumbs may slake or run out to form a shallow heap on the bottom of the beaker, but there is no sign of cloudiness caused by colloidal in suspension.	Non-Dispersive
G2	Slightly Reaction	A very slightly cloudiness can be seen in the water at the surface of the crumb.	Non-Dispersive
G3	Moderate Reaction	There is easily recognizable cloud of colloidal in suspension, usually spreading out in thin streaks at the bottom of the beaker.	Dispersive
G4	Strong Reaction	A colloidal cloud covers most of the bottom of the beaker, usually as a thin skin. In extreme case, all the water becomes cloudy.	Dispersive

AP Engineering & Testing, Inc.

Geotechnical Testing Laboratory

CRUMB TEST RESULTS

Client: Terra Engineers, Inc.

Project Name: Camanche Embankments

Project No.: N/A

AP Job No.: 28-0755

Report Date: 08/04/08

Test Date	**Boring No.**	**Sample No.**	**Depth (feet)**	**Results (G)**			
				Time=1 min	**Time = 5 min**	**Time = 10 min**	**After 10 min**
8/1/2008	DH-204	MC-3A	9.75				
				G1	G2	G3	G4
8/1/2008	DH-204	MC-5A	13.25				
				G1	G1	G2	G3

Grade	Description	Reaction	Categories
G1	No Reaction	Crumbs may slake or run out to form a shallow heap on the bottom of the beaker, but there is no sign of cloudiness caused by colloidal in suspension.	Non-Dispersive
G2	Slightly Reaction	A very slightly cloudiness can be seen in the water at the surface of the crumb.	Non-Dispersive
G3	Moderate Reaction	There is easily recognizable cloud of colloidal in suspension, usually spreading out in thin streaks at the bottom of the beaker.	Dispersive
G4	Strong Reaction	A colloidal cloud covers most of the bottom of the beaker, usually as a thin skin. In extreme case, all the water becomes cloudy.	Dispersive

AP Engineering & Testing, Inc.

Geotechnical Testing Laboratory

CRUMB TEST RESULTS

Client: Terra Engineers, Inc.

Project Name: Camanche Embankments

Project No.: N/A

AP Job No.: 28-0755

Report Date: 08/04/08

Test Date	Boring No.	Sample No.	Depth (feet)	Results (G)			
				Time=1 min	Time = 5 min	Time = 10 min	After 10 min
8/1/2008	DH-205	MC-2A	7.25				
				G1	G2	G3	G4
8/1/2008	DH-205	MC-4	11.25				
				G1	G3	G4	G4

Grade	Description	Reaction	Categories
G1	No Reaction	Crumbs may slake or run out to form a shallow heap on the bottom of the beaker, but there is no sign of cloudiness caused by colloidal in suspension.	Non-Dispersive
G2	Slightly Reaction	A very slightly cloudiness can be seen in the water at the surface of the crumb.	Non-Dispersive
G3	Moderate Reaction	There is easily recognizable cloud of colloidal in suspension, usually spreading out in thin streaks at the bottom of the beaker.	Dispersive
G4	Strong Reaction	A colloidal cloud covers most of the bottom of the beaker, usually as a thin skin. In extreme case, all the water becomes cloudy.	Dispersive

AP Engineering & Testing, Inc.

Geotechnical Testing Laboratory

CRUMB TEST RESULTS

Client:	Terra Engineers, Inc.	AP Job No.:	28-0755
Project Name:	Camanche Embankments	Report Date:	08/04/08
Project No.:	N/A		

Test Date	Boring No.	Sample No.	Depth (feet)	Results (G)			
				Time=1 min	Time = 5 min	Time = 10 min	After 10 min
8/1/2008	DH-206	MC-3A	9.75				
				G1	G2	G4	G4
8/1/2008	DH-206	MC-5	13.25				
				G1	G2	G4	G4

Grade	Description	Reaction	Categories
G1	No Reaction	Crumbs may slake or run out to form a shallow heap on the bottom of the beaker, but there is no sign of cloudiness caused by colloidal in suspension.	Non-Dispersive
G2	Slightly Reaction	A very slightly cloudiness can be seen in the water at the surface of the crumb.	Non-Dispersive
G3	Moderate Reaction	There is easily recognizable cloud of colloidal in suspension, usually spreading out in thin streaks at the bottom of the beaker.	Dispersive
G4	Strong Reaction	A colloidal cloud covers most of the bottom of the beaker, usually as a thin skin. In extreme case, all the water becomes cloudy.	Dispersive

AP Engineering & Testing, Inc.

Geotechnical Testing Laboratory

CRUMB TEST RESULTS

Client: Terra Engineers, Inc.

Project Name: Camanche Embankments

Project No.: N/A

AP Job No.: 28-0755

Report Date: 08/04/08

Test Date	Boring No.	Sample No.	Depth (feet)	Results (G)			
				Time=1 min	Time = 5 min	Time = 10 min	After 10 min
8/1/2008	DH-207	MC-2	7.25				Picture Not Available
				G1	G2	G2	G3
8/1/2008	DH-207	MC-4	11.25				
				G1	G1	G1	G2

Grade	Description	Reaction	Categories
G1	No Reaction	Crumbs may slake or run out to form a shallow heap on the bottom of the beaker, but there is no sign of cloudiness caused by colloidal in suspension.	Non-Dispersive
G2	Slightly Reaction	A very slightly cloudiness can be seen in the water at the surface of the crumb.	Non-Dispersive
G3	Moderate Reaction	There is easily recognizable cloud of colloidal in suspension, usually spreading out in thin streaks at the bottom of the beaker.	Dispersive
G4	Strong Reaction	A colloidal cloud covers most of the bottom of the beaker, usually as a thin skin. In extreme case, all the water becomes cloudy.	Dispersive

AP Engineering & Testing, Inc.

Geotechnical Testing Laboratory

CRUMB TEST RESULTS

Client: Terra Engineers, Inc.

Project Name: Camanche Embankments

Project No.: N/A

AP Job No.: 28-0755

Report Date: 08/04/08

Test Date	Boring No.	Sample No.	Depth (feet)	Results (G)			
				Time=1 min	Time = 5 min	Time = 10 min	After 10 min
8/1/2008	DH-208	MC-2	7.25				
				G1	G1	G2	G3
8/1/2008	DH-208	MC-4	13.25				
				G1	G1	G2	G2

Grade	Description	Reaction	Categories
G1	No Reaction	Crumbs may slake or run out to form a shallow heap on the bottom of the beaker, but there is no sign of cloudiness caused by colloidal in suspension.	Non-Dispersive
G2	Slightly Reaction	A very slightly cloudiness can be seen in the water at the surface of the crumb.	Non-Dispersive
G3	Moderate Reaction	There is easily recognizable cloud of colloidal in suspension, usually spreading out in thin streaks at the bottom of the beaker.	Dispersive
G4	Strong Reaction	A colloidal cloud covers most of the bottom of the beaker, usually as a thin skin. In extreme case, all the water becomes cloudy.	Dispersive

AP Engineering & Testing, Inc.

Geotechnical Testing Laboratory

CRUMB TEST RESULTS

Client:	Terra Engineers, Inc.	AP Job No.:	28-0755
Project Name:	Camanche Embankments	Report Date:	08/04/08
Project No.:	N/A		

Test Date	Boring No.	Sample No.	Depth (feet)	Results (G)			
				Time=1 min	Time = 5 min	Time = 10 min	After 10 min
8/1/2008	DH-209	MC-3	9.75				
				G1	G2	G3	G4
8/1/2008	DH-209	MC-5A	13.25				
				G1	G2	G3	G4

Grade	Description	Reaction	Categories
G1	No Reaction	Crumbs may slake or run out to form a shallow heap on the bottom of the beaker, but there is no sign of cloudiness caused by colloidal in suspension.	Non-Dispersive
G2	Slightly Reaction	A very slightly cloudiness can be seen in the water at the surface of the crumb.	Non-Dispersive
G3	Moderate Reaction	There is easily recognizable cloud of colloidal in suspension, usually spreading out in thin streaks at the bottom of the beaker.	Dispersive
G4	Strong Reaction	A colloidal cloud covers most of the bottom of the beaker, usually as a thin skin. In extreme case, all the water becomes cloudy.	Dispersive

AP Engineering & Testing, Inc.

Geotechnical Testing Laboratory

CRUMB TEST RESULTS

Client: Terra Engineers, Inc.

Project Name: Camanche Embankments

Project No.: N/A

AP Job No.: 28-0755

Report Date: 08/01/08

Test Date	Boring No.	Sample No.	Depth (feet)	Results (G)			
				Time=1 min	Time = 5 min	Time = 10 min	After 10 min
7/31/2008	DH-210	MC-1	5.75				
				G1	G2	G2	G3
7/31/2008	DH-210	MC-4A	11.25				
				G1	G3	G4	G4

Grade	Description	Reaction	Categories
G1	No Reaction	Crumbs may slake or run out to form a shallow heap on the bottom of the beaker, but there is no sign of cloudiness caused by colloidal in suspension.	Non-Dispersive
G2	Slightly Reaction	A very slightly cloudiness can be seen in the water at the surface of the crumb.	Non-Dispersive
G3	Moderate Reaction	There is easily recognizable cloud of colloidal in suspension, usually spreading out in thin streaks at the bottom of the beaker.	Dispersive
G4	Strong Reaction	A colloidal cloud covers most of the bottom of the beaker, usually as a thin skin. In extreme case, all the water becomes cloudy.	Dispersive

AP Engineering & Testing, Inc.

Geotechnical Testing Laboratory

CRUMB TEST RESULTS

Client:	Terra Engineers, Inc.	AP Job No.:	28-0755
Project Name:	Camanche Embankments	Report Date:	08/01/08
Project No.:	N/A		

Test Date	Boring No.	Sample No.	Depth (feet)	Results (G)			
				Time=1 min	Time = 5 min	Time = 10 min	After 10 min
7/31/2008	DH-211	MC-3A	9.25				
				G1	G3	G4	G4
7/31/2008	DH-211	MC-5A	13.75	Picture Not Available			
				G1	G3	G4	G4

Grade	Description	Reaction	Categories
G1	No Reaction	Crumbs may slake or run out to form a shallow heap on the bottom of the beaker, but there is no sign of cloudiness caused by colloidal in suspension.	Non-Dispersive
G2	Slightly Reaction	A very slightly cloudiness can be seen in the water at the surface of the crumb.	Non-Dispersive
G3	Moderate Reaction	There is easily recognizable cloud of colloidal in suspension, usually spreading out in thin streaks at the bottom of the beaker.	Dispersive
G4	Strong Reaction	A colloidal cloud covers most of the bottom of the beaker, usually as a thin skin. In extreme case, all the water becomes cloudy.	Dispersive

AP Engineering & Testing, Inc.

PINHOLE DISPERSION TEST
ASTM D 4647

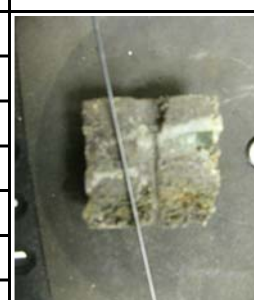
Project Name:	Camanche Embankments	Tested by KK/KM	Date 08/27/08
Project No.:	NA	Calculated by KM	Date 08/28/08
Boring No.:	DH-203	Checked by AP	Date 08/28/08
Sample No.:	MC-2A Depth: 7.25 feet		
Soil Description:	Brownish Gray Clayey Sand		

INITIAL CONDITION OF SPECIMEN

Diameter (d)	1.36	in		Before	After
Sample Area (A)	1.45	in²	Container No.		
Length (L)	1.50	in	Wt. Wet Soil+Container(g)	436.35	129.65
Weight Before	81.29	g	Wt. Dry Soil+Container(g)	412.82	120.20
Wet Density	142.3	pcf	Wt. Container (g)	197.2	49.88
Dry Density	128.3	pcf	Moisture, (%)	10.91	13.44

Pinhole Size Before Test 0.039 inches

TEST DATA

Clock Time	Elapsed Time (s)	Hydraulic Head (in)	Effluent (ml)	Flow Rate (ml/s)	Cloudiness of Effluent	Method A Classification	Specimen After Test
15:00	10	2	5	0.50	Barely Visible		
	10	2	5	0.50	Barely Visible		
	10	2	5	0.50	Barely Visible		
	65	2	25	0.38	Barely Visible		
	126	2	50	0.40	Barely Visible		
	253	2	100	0.40	Barely Visible		
	60	2	30	0.50	Barely Visible		
	300	2	110	0.37	Barely Visible		
	600	2	217	0.36	Barely Visible		
	300	7	250	0.83	Barely Visible		
	300	15	530	1.77	Barely Visible		
	300	40	830	2.77	Barely Visible	**ND1**	

Remarks: Pinhole size after test is about the same as needle diameter.

AP Engineering & Testing, Inc.

PINHOLE DISPERSION TEST

ASTM D 4647

Project Name: Camanche Embankments | Tested by KK/KM Date 08/25/08
Project No.: NA | Calculated by KM Date 08/26/08
Boring No.: DH-206 | Checked by AP Date 08/26/08
Sample No.: MC-5 Depth: 13.25 feet
Soil Description: Brown Sandy Clay

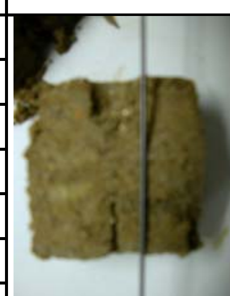
INITIAL CONDITION OF SPECIMEN

Diameter (d)	1.36	in
Sample Area (A)	1.45	in²
Length (L)	1.50	in
Weight Before	76.55	g
Wet Density	134.0	pcf
Dry Density	120.4	pcf

	Before	After
Container No.		
Wt. Wet Soil+Container(g)	523.32	127.62
Wt. Dry Soil+Container(g)	490.19	118.96
Wt. Container (g)	197.38	50.25
Moisture, (%)	11.31	12.60

Pinhole Size Before Test 0.039 inches

TEST DATA

Clock Time	Elapsed Time (s)	Hydraulic Head (in)	Effluent (ml)	Flow Rate (ml/s)	Cloudiness of Effluent	Method A Classification	Specimen After Test
16:00	10	2	3	0.30	Clear		
	10	2	3	0.30	Clear		
	10	2	3	0.30	Clear		
	90	2	25	0.28	Clear		
	210	2	50	0.24	Clear		
	435	2	100	0.23	BarelyVisible		
	60	2	13	0.22	BarelyVisible		
	300	2	73	0.24	BarelyVisible		
	600	2	143	0.24	BarelyVisible		
	300	7	250	0.83	BarelyVisible		
	300	15	400	1.33	BarelyVisible		
	300	40	780	2.60	BarelyVisible	**ND1**	

Remarks: Pinhole size after test is about the same as needle diameter.

AP Engineering & Testing, Inc.

PINHOLE DISPERSION TEST
ASTM D 4647

Project Name: Camanche Embankments
Project No.: NA
Boring No.: DH-207
Sample No.: MC-2 Depth: 7.25 feet
Soil Description: Brown Silty Clay with fine sand

Tested by KK/KM Date 08/26/08
Calculated by KM Date 08/28/08
Checked by AP Date 08/28/08

INITIAL CONDITION OF SPECIMEN

Diameter (d)	1.36	in
Sample Area (A)	1.45	in²
Length (L)	1.50	in
Weight Before	66.73	g
Wet Density	116.8	pcf
Dry Density	90.6	pcf

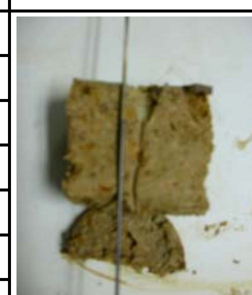
	Before	After
Container No.		
Wt. Wet Soil+Container(g)	465.25	116.44
Wt. Dry Soil+Container(g)	404.88	101.04
Wt. Container (g)	196.15	49.71
Moisture, (%)	28.92	30.00

Pinhole Size Before Test 0.039 inches

TEST DATA

Clock Time	Elapsed Time (s)	Hydraulic Head (in)	Effluent (ml)	Flow Rate (ml/s)	Cloudiness of Effluent	Method A Classification
14:00	10	2	5	0.45	Clear	
	10	2	5	0.50	Clear	
	10	2	5	0.50	Clear	
	71	2	25	0.35	Clear	
	150	2	50	0.33	Clear	
	327	2	100	0.31	BarelyVisible	
	60	2	19	0.32	BarelyVisible	
	300	2	77	0.26	BarelyVisible	
	600	2	78	0.13	BarelyVisible	
	300	7	210	0.70	BarelyVisible	
	300	15	396	1.32	BarelyVisible	
	300	40	660	2.20	BarelyVisible	**ND1**

Specimen After Test



Remarks: Pinhole size after test is about the same as needle diameter.

AP Engineering & Testing, Inc.

PINHOLE DISPERSION TEST
ASTM D 4647

Project Name:	Camanche Embankments	Tested by KK/KM	Date 08/26/08
Project No.:	NA	Calculated by KM	Date 08/28/08
Boring No.:	DH-211	Checked by AP	Date 08/28/08
Sample No.:	MC-3A Depth: 9.25 feet		
Soil Description:	Olive Brown Clayey Sand		

INITIAL CONDITION OF SPECIMEN

Diameter (d)	1.36	in
Sample Area (A)	1.45	in²
Length (L)	1.50	in
Weight Before	79.20	g
Wet Density	138.7	pcf
Dry Density	124.9	pcf

	Before	After
Container No.		
Wt. Wet Soil+Container(g)	301.85	128.45
Wt. Dry Soil+Container(g)	287.08	120.20
Wt. Container (g)	153.35	50.46
Moisture, (%)	11.04	11.83

Pinhole Size Before Test 0.039 inches

TEST DATA

Clock Time	Elapsed Time (s)	Hydraulic Head (in)	Effluent (ml)	Flow Rate (ml/s)	Cloudiness of Effluent	Method A Classification
9:00	10	2	5	0.50	BarelyVisible	
	10	2	4	0.40	BarelyVisible	
	10	2	6	0.60	BarelyVisible	
	55	2	25	0.45	BarelyVisible	
	111	2	50	0.45	BarelyVisible	
	224	2	100	0.45	BarelyVisible	
	60	2	26	0.43	BarelyVisible	
	300	2	126	0.42	BarelyVisible	
	600	2	253	0.42	BarelyVisible	
	300	7	260	0.87	BarelyVisible	
	300	15	450	1.50	BarelyVisible	
	300	40	785	2.62	BarelyVisible	**ND1**

Specimen After Test

Remarks: Pinhole size after test is about the same as needle diameter.

ENCLOSURE C
PHASE II FIELD INVESTIGATION AND LABORATORY TESTING

CHARACTERIZATION OF CORE MATERIALS AND FOUNDATION TAILINGS AT MAIN DAM

CAMANCHE EMBANKMENTS SAFETY REVIEW

PHASE II FIELD INVESTIGATION AND LABORATORY TESTING

CHARACTERIZATION OF CORE MATERIALS AND FOUNDATION TAILINGS AT MAIN DAM

DATA REPORT

Prepared for

EAST BAY MUNICIPAL UTILITY DISTRICT
375 Eleventh Street
Oakland, CA 94607

March 2009

TERRA / GeoPentech
a Joint Venture

Black & Veatch
AGS
Natural Resources Consulting Engineers
AP Engineering and Testing

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SECTION 1.0 INTRODUCTION

A Field Exploration and Laboratory Testing program was conducted on the main dam of the Camanche Reservoir between September and November, 2008 as part of the Camanche Embankments Safety Review. This program facilitated the collection of field and laboratory data to supplement the existing subsurface information from construction records and previous studies. The data presented in this report will be utilized in characterizing the properties of the embankment and foundation materials for use in the analyses of the main dam. The characterization of the materials will be reported in future technical memoranda along with the results of the particular type of analyses being completed.

The field and laboratory portions of the program are described in Sections 2 and 3 of this report, respectively. There are three appendices to this report containing the following: Boring Logs (Appendix A), results of Cone Penetration Testing (Appendix B) and Laboratory Test Results (Appendix C).

The preparation of this report and the work described herein was performed by Terra/GeoPentech, a Joint Venture (Terra/GeoPentech) as part of our Camanche Embankments Safety Review contract with the East Bay Municipal Utility District (District).

2.1 PURPOSE AND SCOPE

The purpose of the field program was to collect samples and in-situ data of embankment and foundation materials from the main dam. These data will be used in concordance with existing data to characterize the dam material properties and conduct detailed seepage and stability analyses. Following a comprehensive review of the existing data, the exploration program was prepared which specifically targeted the core and tailings materials at the downstream toe of the main dam.

The scope of the field exploration program consisted of the drilling and sampling of six (6) rotary wash borings with subsequent piezometer installation, and completion of ten (10) Cone Penetration Tests (CPT) soundings using a seismic cone. The locations of these borings and CPT soundings are shown on Figure 1. Further descriptions of the drilling, piezometer installation and CPT soundings are presented below.

2.2 DRILLING

The drilling of all borings was performed by PC Exploration, Inc. of Rocklin, California. The drilling operations were performed under full-time supervision and direction of Mr. Richard Harlan, CEG. The materials recovered from the borings were visually classified and logged in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D-2488-06. Table 1 presents a summary of the boring and piezometer locations and installation details; the boring logs are presented in Appendix A.

All borings were drilled with a Mobile B-80 rotary drill rig. The borings were initially drilled with 8-inch diameter hollow stem augers to advance and case the holes through the surface layer of gravel and cobble fill (embankment slope protection, Zone 6) placed at the crest and toe of the dam.

Crest boring MB-1 was initially advanced via augers to a depth of 48 feet; the augers were then pulled and replaced with 5-inch diameter flush joint casing set to 49 feet, prior to converting to rotary wash, in order to prevent rotary drill fluid pressures from potentially affecting the narrower upper portion of the core. MB-1 was then advanced via rotary wash drilling to a depth of 193 feet (11 feet below the contact of the Zone 1 core and Mehrten foundation at approximately 182 feet), using a 4-inch carbide “blade” bit and biodegradable Revert-based drill fluid. MB-1 was completed to its total depth of 220 feet, through the Mehrten Formation, via HQ coring using a double core barrel with a solid inner-tube. No fluid losses occurred while advancing MB-1 through the Zone 1 core portion of the embankment. However, a near total fluid loss occurred while coring through a section of the Mehrten foundation when the boring was at a depth of approximately 205 feet; this loss may have been concentrated along a fine-grained sandstone layer encountered at a depth of 197 to 199.5 feet, or within a zone of fracturing between 203 and 204.5 feet. Both rotary and core drilling operations utilized a Moyno centrifugal-type pump to eliminate the potential for pump surging and attendant potential high fluid pressures that could result from using a conventional piston-type pump. Drill fluid pressures were continually monitored at the surface via a pressure gauge installed on the drill rig at the downstream end of the pump.

Alternating Standard Penetration Test (SPT) and undisturbed Shelby tube sampling was performed during the course of drilling the Zone 1 core in boring MB-1; samples were driven or pushed at typically 5-foot intervals. AWJ-size rods were used for the SPT sampling in MB-1 above a depth of 50 feet; NWJ-size rods were used below that depth. As mentioned above, the Mehrten foundation in MB-1 was sampled via HQ coring below a depth of 193 feet. Photographs of the HQ core recovered from boring MB-1 are shown on Figure 2.

The downstream toe borings (MB-2 through MB-5) encountered the Zone 6 surface gravels and cobbles to depths ranging from 3.5 to 17 feet, with the deeper extent of gravels occurring towards the north side of the channel area. An initial attempt to advance boring MB-5 via 8-inch auger drilling encountered refusal within the approximately 17-foot-thick zone of gravels and cobbles that occurs at that location, and the boring was abandoned (that abandoned boring was designated MB-5x). MB-5 was completed later at a location 5 feet to the north of its original location using a 5-inch diameter pneumatic rotary-percussion hammer (ODEX system) to advance the hole through the cobbles. Below their surface-cased sections (augers or ODEX), the toe borings were completed via rotary wash drilling with Revert-based fluid.

Near-continuous SPT sampling was performed in the tailings portions of the downstream toe borings in general accordance with ASTM D-6066-96 (2004). Each boring was drilled out for a distance of one foot below the bottom of each 18-inch drive, prior to subsequent SPT sampling. SPT testing in the toe borings utilized a standard 2-in. outside diameter, 1.4-in. inside diameter sampler, with NWJ-size rods, and an automatic 140 lb. hammer with 30-in. drop. Mr. Virgil Baker of Gregg Drilling and Testing, Inc. performed energy-calibration measurements of the hammer mechanism during SPT sampling of MB-3. Results of this calibration are presented in Enclosure 1 of Appendix A.

2.3 INSTALLATION OF PIEZOMETERS

The installation of all piezometers was performed by PC Exploration, Inc. of Rocklin, California. The operation was performed under full-time supervision and direction of Mr. Richard Harlan, CEG. Table 1 is a summary of the boring locations with piezometer installation details. Table 2 presents data on the initial measurements, including check readings taken immediately prior to installation and the initial readings taken after installation.

A multi-level array of 4 grouted-in vibrating wire piezometers (VWPs) was installed in boring MB-1. Upon completion of the drilling, the boring was flushed with a thick Revert-based fluid to further remove cuttings and stabilize the borehole prior to installation. The four VWP signal cable spools were unwound along the crest of the dam, and both dry and saturated frequency readings were taken for each of the VWPs to check their functioning prior to being installed in the borehole. VWP check readings taken prior to installation and initial readings taken after installation are presented in Table 2. Each VWP was attached with Duct-tape onto the side of a 1¼-inch diameter Schedule 40 PVC pipe, at a pre-determined depth, and the signal cable was then strung up into the PVC via a hole drilled into the pipe approximately one foot below the attached VWP so that the PVC pipe functioned as both a protective conduit for the signal cables and a grout pipe to tremie-backfill the VWPs into the borehole. The piezometer cables were strung through each 10-foot length of PVC, at the surface, as the assembled array was lowered into the hole. The final piezometer depths were at 207.3 feet (in the Mehrten foundation), 170.3 feet, 120.3 feet, and 70.3 feet (all within the embankment core). These installed depths were 4.7 feet higher than the original target depths because some caving or cuttings settlement in-filled the bottom section of the hole during installation.

The installed VWP array was then grouted in-place using a grout mix as recommended by Slope Indicator Company for such multiple VWP installations that was comprised of 94 lbs of Portland cement for 30 gallons of water and 25 lbs of bentonite. The specified grout mix was pumped down the 1¼-inch PVC pipe, via the tremie-method, using a special "Y" fitting at the top of the grout/cable pipe. This fitting allowed connection of the grout line at one port, with the second port closing off the top of the pipe where the VWP signal cables exit the hole. The installation and grouting of the VWP array in MB-1 was observed by representatives of the California Department of Water Resources Division of Dam Safety (Messrs. Bill Fraser and Jim Lessman).

Following installation and grouting of the multi-level VWP in MB-1, a shallow trench was hand excavated to route the VWP cables to a terminal box mounted on the guardrail along the downstream side of the crest roadway. The 6-inch wide trench was excavated to a depth of approximately 10 inches. The four VWP cables were strung through a protective 1¼-inch PVC pipe that connected with an elbow to the main, vertical grout tremie/cable conduit pipe. The horizontal section of cable conduit was then placed along the bottom of the trench and covered with a sand backfill to a depth of approximately six inches. The remaining upper portion of the trench was backfilled to the surface with Zone 6 gravel and cobbles that were tamped in-place. The roadway portion of the trench and top of the boring were then filled with an asphalt patch. The final section of cables was also routed though PVC that was cemented to a fitting on the bottom of the terminal box, such that the entire length of signal cables is protected within PVC conduit.

The VWP piezometer cables from MB-1 were wired into the terminal box that was mounted on the wooden guardrail support post immediately downstream of the boring. The signal cable from the deepest piezometer, MB-1-1 at a depth of 207.3 feet, was wired into terminal box switch No. 1; MB-1-2 at 170.3 ft. was wired into switch No. 2; MB-1-3 at 120.3 ft. into switch No. 3, and MB-1-4 at 70.3 ft. was wired into switch No. 4. After the cables were wired into the terminal box, frequency readings were again taken on October [1], 2 and 3, 2008. As shown on Table 2, the October 3rd reading of the MB-1-1 piezometer (installed in the Mehrten foundation below the embankment core at a depth of 207 feet) was consistent with the previous readings taken on September 29th, suggesting that the piezometric level in the foundation had stabilized at that VWP sensing zone. The October 3rd readings of the other three VWPs (installed at various levels in the embankment core) showed a continuing gradual decline in head from previous readings, indicating that the piezometric levels in the core were still reflecting drill fluid effects and had not yet stabilized as of that date.

The installation of the single vibrating wire piezometer in MB-3vwp was performed according to a method similar to that used for the MB-1 installation. The VWP was taped to a 1¼-inch PVC pipe that was used as a combination grout pipe and signal cable conduit, and tremie-grouted in place at a depth of 13 feet with the same cement/water/bentonite mix as used for MB-1. This installation was completed with the upper end of the signal cable left coiled within a protective steel surface casing, and with the ends of the individual readout wires from the cable left wrapped in electrical tape for protection. As shown on Table 2, initial frequency readings of MB-3vwp were taken just prior to, and after, installation.

Each of the conventional piezometers installed in borings MB-2 through MB-5 consists of a "Casagrande-type" porous plastic-tipped piezometer (one foot in length), attached to the bottom of a 1-inch diameter Schedule 40 PVC standpipe riser. The bottom of the tip was placed in the lower tailings section, typically about one foot above the tailings/Mehrten contact, and a sand backfill (Lapis Lustre #3) was placed beneath and around the tip, and up to a height of approximately one foot above the tip. A two-foot bentonite seal was placed above the sand pack, and the hole backfill then finished with a cement grout seal placed to the surface via the tremie method. A 9-inch diameter protective steel surface casing was installed over each of the PVC standpipes.

2.4 CPT SOUNDINGS

The CPT soundings were performed by Gregg Drilling and Testing, Inc., of Martinez, California. A 30-ton truck-mounted CPT rig was used to continuously advance the cone through the main dam embankment core (Zone 1), and through the tailings or alluvial deposits that comprise the shell foundation at the downstream toe of the dam. Four-inch diameter starter holes were advanced (via a truck mounted auger drill) through the layer of gravels and cobbles (Zone 6) that overlie both the core at the crest of the dam and the tailings at the downstream toe. Pore water dissipation and seismic velocity tests were conducted at selected intervals. On completion of each sounding, the cone hole was tremie-grout backfilled through CPT rods that were re-inserted into the hole. Table 3 presents details of individual CPT soundings. The results of these soundings are contained in Appendix B.

The depths shown on the CPT logs are the nominal depths based on the length of the rods that push the CPT probe. Deviations from vertical cause the actual depths to be different from the nominal depths – usually by an insignificant amount. However, deviations from vertical for CPT MC-7 were such that a depth correction of 10.3 feet was necessary at the bottom. The maximum depth correction at all other CTP locations was 2 feet or less. These depth corrections were made when analyzing the data but have not been made on the logs contained in Appendix B.

CPT soundings were completed at ten (10) of thirteen (13) originally planned locations. The CPT met refusal in deep cobbles at the planned locations of MC-5, MC-12 and MC-13. Five of the completed CPTs were performed from the crest of the main dam, and were pushed through the entire section of Zone 1 core material to refusal in the underlying Mehrten foundation (MC-1, and MC-6 through 9). The remaining five completed CPTs were located along the downstream toe in the southern portion of the channel area, and were pushed through mine tailings or alluvium, also to refusal in Mehrten Formation (MC-2, 3, 4, 10 and 11).

2.5 FIELD OBSERVATIONS

The core materials usually vary from silt, sandy silt, silty sand, sandy clay, and clayey sand, with non-plastic to moderately-plastic fines. One apparently thin layer of clean fine sand (SP) was noted at a depth of 93 feet. Occasional gravelly zones were encountered between approximately 133 and 168 feet. No indications of desiccation cracking, or other open cracks or seams, were noted within the Zone 1 core materials encountered in the MB-1 samples. As mentioned previously, no fluid losses occurred while advancing MB-1 through the core portion of the embankment.

The tailings materials encountered at the downstream toe were observed to be fine, poorly-graded sand (SP) and silty sand (SM), and typically loose to very loose, and locally medium dense. Some finer deposits were encountered, usually as thin interbeds of non-plastic to low-plasticity silt and sandy silt. Locally thicker deposits of soft, low to moderate plasticity silt, sandy silt and lean clay were encountered in boring MB-4. Mehrten formation siltstone was encountered underlying the tailings at depths ranging from 14 to 27 feet, with the deeper contacts encountered towards the present Mokelumne River channel.

3.1 PURPOSE AND SCOPE

The laboratory testing program was focused on assisting in the classification and characterization of embankment and foundation materials. Classification tests were performed to support the interpretation of CPT and SPT data for both core and tailings materials. Strength testing was performed on undisturbed samples of core material to facilitate the development of strength properties for the dam core.

The test results are presented in Appendix C, the results of the index tests are also included on the boring logs at the appropriate depths. Table C-1 provides a summary of the laboratory test results and is followed by detailed results for all tests performed. As shown on Table C-1, the scope of the laboratory testing included:

- Visual classification on all samples in accordance with ASTM D-2488
- Twenty-three moisture content measurements in accordance with ASTM D-2216
- Fifteen dry unit weight measurements in accordance with D-2937
- Twenty-seven particle size distribution analyses by sieving and hydrometer in accordance with ASTM D-422
- Twenty-four determinations of percent passing the No. 200 sieve in accordance with ASTM D-1140
- Forty-five Atterberg Limits tests in accordance with ASTM D-4318
- Fourteen consolidated undrained triaxial tests with pore pressure measurements performed in accordance with ASTM D-4767

3.2 CLASSIFICATION TESTING

Classification tests were performed on bag and Shelby tube samples obtained from the rotary wash borings as described above. The primary focus of this portion of the program was to assist in the interpretation of CPT results and SPT data recorded during the field exploration. The secondary focus was to determine which undisturbed Shelby tube samples would appropriately represent the core material for the strength testing portion of the program.

All fifteen SPT samples of core material were tested for Atterberg Limits and percent passing the No. 200 sieve. This provided the information required to perform an SPT-based liquefaction triggering analysis for the core material. Similarly, SPT samples of tailings material were tested to determine percent passing No. 200 sieve. Subsequently, samples with more than 10% fines content were tested for Atterberg Limits. This provided the information required to perform an SPT-based liquefaction triggering analysis and to develop an SPT-based residual strength ratio.

In addition to classifying SPT samples, fourteen Shelby tubes of core material which had been sampled below the phreatic surface in boring MB-1 were selected for classification testing. Material was removed from the bottom tip of each tube and moisture content, particle size distribution and Atterberg Limits were determined.

3.3 STRENGTH TESTING

The primary goal of the strength testing program was to obtain sufficient data for the development of undrained strength parameters for the core material. Undrained shear strength will be used in the static and dynamic stability analyses to follow.

After visual inspection of the samples and review of classification testing results, nine Shelby tube samples from MB-1 were selected for consolidated undrained triaxial tests (TX-CIU’). In each instance, only the required material was removed from the Shelby tube and the tubes were then re-sealed and saved for further possible testing. These tests were performed with a confining pressure that was slightly higher that the in-situ confining pressure of the sample ($\sigma_{vc}'_{(tested)}$ / $\sigma_{vc}'_{(in\text{-}situ)}$ ~ 1.1 to 1.2). The confining pressures ranged from 6.5 to 15.8 kips per square foot (ksf).

Following the completion of these nine tests and a preliminary review of the results, five Shelby tube with sufficient undisturbed material remaining were selected for an additional TX-CIU’ test. In this second round of testing, higher confining pressures were applied to the samples ($\sigma_{vc}'_{(tested)}$ / $\sigma_{vc}'_{(in\text{-}situ)}$ ~ 1.4 to 1.8), to further understand the behavior of the core materials and to assist in the interpretation of the data. After the completion of the TX-CIU’ test, each of the fourteen samples was tested for Atterberg Limits and grain size distribution. The tabulated results for each TX-CIU’ test are included in Appendix C, along with a stress vs. axial strain plot, change in pore water pressure vs. axial strain plot, p-q’ plot, and Mohr’s circle plot.

TABLES

TABLE 1

MAIN DAM BORINGS, PIEZOMETER LOCATIONS AND INSTALLATION DETAILS, 2008

Boring/Piezometer No. (VWP Serial No.)	Piezometer Depth, ft. (Elevation)	Main Dam Surface Location and Elevation	Sensing Zone Depth, ft	Sensing Zone Location
MB-1-1* (96624)	207.3 (El. 56)	Crest Station 314+87 El. 263	207.3	Mehrten Fm. (foundation)
MB-1-2* (96819)	170.3 (El. 93)	Crest Station 314+87 El. 263	170.3	Embankment Core
MB-1-3* (96293)	120.3 (El. 143)	Crest Station 314+87 El. 263	120.3	Embankment Core
MB-1-4* (96730)	70.3 (El. 193)	Crest Station 314+87 El. 263	70.3	Embankment Core
MB-2**	18.2-19.2 (El. 91’)	D/S Toe at Sta. 313+00 El. 110’	17-21	Downstream Toe - Tailings
MB-3**	12.2-13.2 (El. 91.5’)	D/S Toe at Sta. 315+00 El. 104.5’	10.8-16	Downstream Toe - Tailings
MB-3 VWP* (96528)	13 (El. 91’)	D/S Toe at Sta. 315+08 El. 104.5’	13	Downstream Toe - Tailings
MB-4**	20.2-21.2 (El. 89’)	D/S Toe at Sta. 316+46 El. 110’	12-23.5	Downstream Toe - Tailings
MB-5**	24.7-25.7 (El. 89’)	D/S Toe at Sta. 319+94 El. 115’	23-28.5	Downstream Toe - Tailings

* Vibrating wire piezometer.

** Porous plastic (Casagrande-type) standpipe piezometer.

TABLE 2

VIBRATING WIRE PIEZOMETER INITIAL FREQUENCY READINGS
MAIN DAM – SEPTEMBER 25, 29 AND OCTOBER 3, 2008

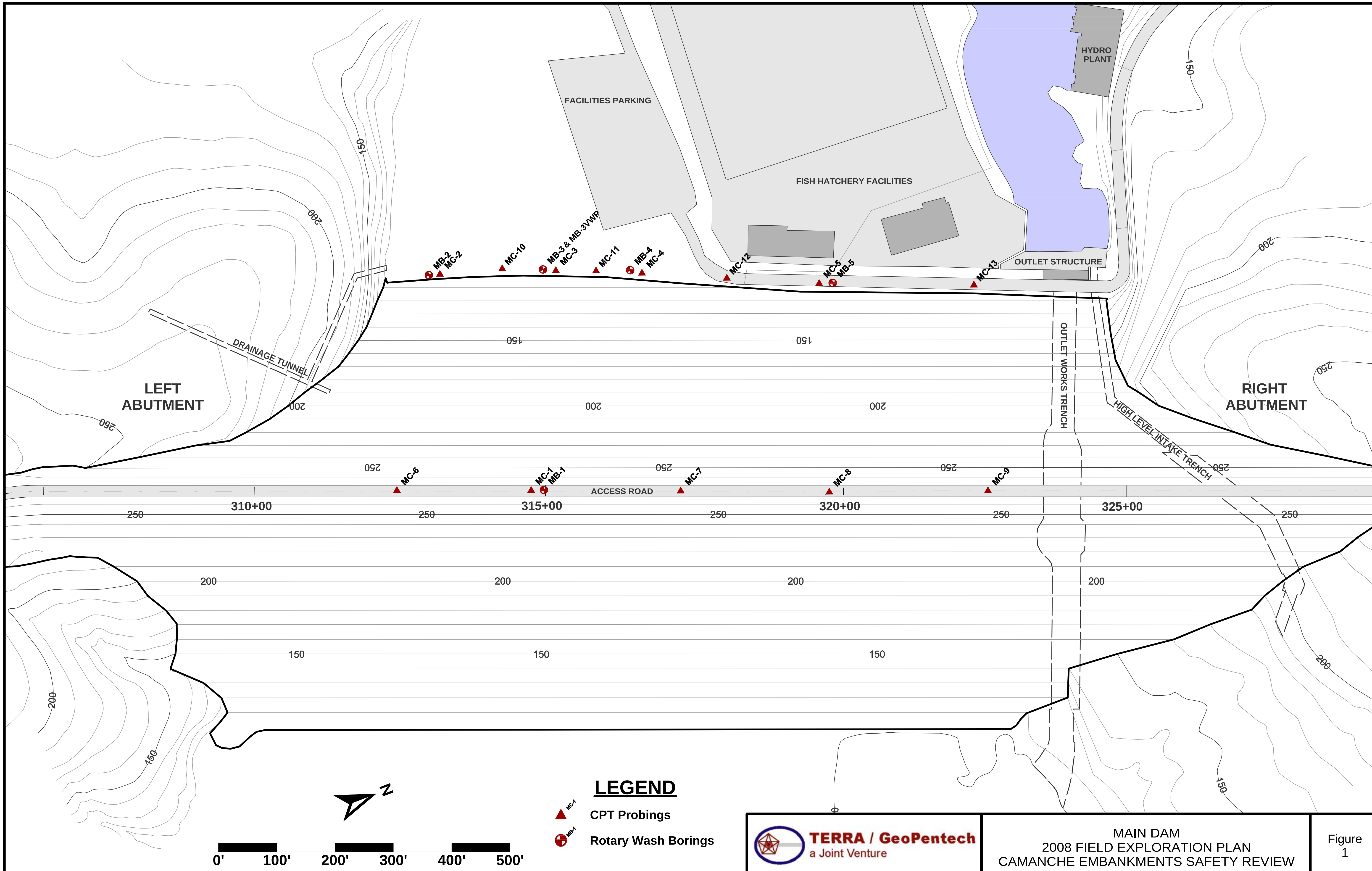
Vibrating Wire Piezometer (Serial Number)	VWP tip depth, ft. (location)	Pre-installation Frequency Readings, Hz (VWP at surface)		Initial Installed Frequency (grouted-in)	September 29, 2008 Readings			October 3, 2008 Frequency, Hz
					Frequency, Hz	Calculated		
		Dry	Wet			Pressure Head, psi	Water Depth, ft. (Elev.)	
MB-1-1 (96624)	207.3 (main dam foundation)	2786.95	2783.41	2550.31 (5:30 pm, 9/25/08)	2552.51	32	133 (El. 130)	2551.92
MB-1-2 (96819)	170.3 (main dam core)	2925.71	2921.6	2591.00 (5:30 pm, 9/25/08)	2608.34	31	98 (El. 165)	2615.45
MB-1-3 (96293)	120.3 (main dam core)	2903.59	2898.33	2670.28 (5:30 pm, 9/25/08)	2693.22	17	81 (El. 182)	2699.09
MB-1-4 (96730)	70.3 (main dam core)	2753.97	2749.93	2703.56 (5:30 pm, 9/25/08)	2716.28	3	63 (El. 200)	2719.55
MB-3 VWP (96528)	13 (D/S toe tailings)	2934.43	2930.96	2876.94 (8:30 am, 10/3/08)	-	-	-	2876.94

TABLE 3

MAIN DAM CPT SOUNDING DETAILS, 2008

CPT Sounding No.	Main Dam Location and Elevation	Materials Encountered	Interpreted Top of Mehrten Fm. depth, ft.	Total Depth at refusal, ft.
MC-1	Crest Station 314+87 El. 263	Embankment Core, Mehrten Fm.	187.9	188.4
MC-2	D/S Toe Sta. 313+00 El. 110’	Tailings, Mehrten Fm.	20.1	21.2
MC-3	D/S Toe Sta. 315+00 El. 104.5’	Tailings, Mehrten Fm.	15.1	18.9
MC-4	D/S Toe Sta. 316+46 El. 110’	Tailings, Mehrten Fm.	24.7	25.3
MC-5	D/S Toe Sta. 319+94 El. 115’	refused in surface cobbles (fill)	-	-
MC-6	Crest Station 312+49 El. 263	Embankment Core, Mehrten Fm.	187.0	187.2
MC-7	Crest Station 317+40 El. 263	Embankment Core, Mehrten Fm.	180.8	180.8
MC-8	Crest Station 319+75 El. 263	Embankment Core, Mehrten Fm.	179.5	181.3
MC-9	Crest Station 322+50 El. 263	Embankment Core, Mehrten Fm.	151.1	161.1
MC-10	D/S Toe Sta. 314+60 El. 107’	Tailings, Mehrten Fm.	17.5	18.2
MC-11	D/S Toe Sta. 316+05 El. 107’	Tailings, Mehrten Fm.	15.1	18.0
MC-12	D/S Toe Sta. 317+60 El. 115’	refused in surface cobbles (fill)	-	-
MC-13	D/S Toe Sta. 322+50 El. 115’	refused in surface cobbles (fill)	-	-

FIGURES



HYDRO PLANT
FACILITIES PARKING
FISH HATCHERY FACILITIES
OUTLET STRUCTURE
LEFT ABUTMENT
RIGHT ABUTMENT
DRAINAGE TUNNEL
OUTLET WORKS TRENCH
HIGH LEVEL INTAKE TRENCH
ACCESS ROAD
MB-2
MC-2
MC-10
MB-3 & MB-3VWP
MC-3
MC-11
MB-4
MC-4
MC-12
MC-5
MB-5
MC-13
MC-6
MC-1
MB-1
MC-7
MC-8
MC-9
310+00
315+00
320+00
325+00
150
200
250
LEGEND
CPT Probings
Rotary Wash Borings
0'
100'
200'
300'
400'
500'
N
TERRA / GeoPentech
a Joint Venture
MAIN DAM
2008 FIELD EXPLORATION PLAN
CAMANCHE EMBANKMENTS SAFETY REVIEW
Figure 1

MB-1, Core Box 1, 193.8 - 204.0 ft.

MB-1, Core Box 2, 204.0 - 220.0 ft.

TERRA / GeoPentech a Joint Venture	BORING MB-1 HQ CORE RECOVERY PHOTOS CAMANCHE EMBANKMENTS SAFETY REVIEW	Figure 2

APPENDIX A

CAMANCHE EMBANKMENTS SAFETY REVIEW

Key to Log of Boring

Sheet 1 of 2

Elevation, feet	Depth, feet	SAMPLES: Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15

COLUMN DESCRIPTIONS

1. **Elevation:** Elevation in feet referenced to specified datum.
2. **Depth:** Depth in feet below the ground surface.
3. **Sample Type:** Type of soil or rock sample collected at depth interval shown; sampler symbols are explained below.
4. **Sample Number:** Sample identification number.
5. **Blows per 6 inches:** Number of blows required to advance sampler each 6-inch drive interval, or distance noted, using a 140-lb hammer dropped 30 inches.
6. **Recovery:** Length in inches of material recovered in sampler.
7. **Graphic Log:** Graphic depiction of subsurface material encountered. Typical symbols are given below; variations on these symbols are used to indicate secondary soil components.
8. **Material Description:** Description of material encountered; in addition to soil or rock classification, may include color, moisture, relative density/consistency, particle size, and plasticity for soil; texture, weathering, strength, and hardness of bedrock.
9. **% Gravel:** Percent of soil by weight retained on the No. 4 sieve as determined per ASTM Method D422.
10. **% Fines:** Percent of soil by weight passing the No. 200 sieve as determined per ASTM Method D422.
11. **Liquid Limit:** Liquid Limit (LL) of soil specimen passing the No. 40 sieve as determined per ASTM Method D4318.
12. **Plasticity Index:** Plasticity Index (PI=LL-PL) of soil specimen passing No. 40 sieve as determined per ASTM Method D4318.
13. **Moisture Content:** Moisture content, as a percentage of dry weight of specimen, determined per ASTM Method D2216.
14. **Dry Unit Weight:** Dry weight per unit volume of soil, reported in pounds per cubic foot, determined per ASTM Method D2937.
15. **Field Notes and Other Tests:** Comments and observations regarding drilling or sampling made by driller or field personnel. Lab test results other than those listed in columnar format may be recorded using abbreviations below.

TYPICAL MATERIAL GRAPHIC SYMBOLS

POORLY GRADED SAND (SP)

SILT (ML)

LEAN CLAY (CL)

POORLY GRADED GRAVEL (GP)

WELL-GRADED SAND with SILT (SW-SM)

SANDY SILT (ML)

SILTY CLAY (CL-ML)

WELL-GRADED GRAVEL (GW)

SILTY SAND (SM)

SILTY, CLAYEY SAND (SC-SM)

CLAYEY SAND (SC)

SILTY GRAVEL (GM)

SANDSTONE

SILTY SANDSTONE

SILTSTONE

CLAYSTONE

TYPICAL SAMPLER GRAPHIC SYMBOLS

Standard Penetration Test (SPT) split spoon

HQ rock core barrel

Shelby tube (thin-wall, fixed-head undisturbed)

Modified California (2.4-inch-ID, 3-inch-OD)

Pitcher Barrel (lined with Shelby tube)

Bulk sample collected from auger cuttings

OTHER LABORATORY TEST ABBREVIATIONS

CONS One-Dimensional Consolidation Test
CRUMB Crumb Test
TX-UU Unconsolidated Undrained Triaxial Compression Test
TX-CIU Isotropically Consolidated Undrained Triaxial Test
UCS Unconsolidated Compressive Strength Test
%<5μ Hydrometer Test (coupled with Sieve Test)

OTHER GRAPHIC SYMBOLS

First water encountered at time of drilling and sampling

Static water level measured at specified time after drilling

Change in material properties within a lithologic unit

— — — Inferred contact between soil strata or gradational change

GENERAL NOTES

1. Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.

2. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO DESCRIPTIVE TERMS FOR ROCK

ROCK WEATHERING / ALTERATION

Description	Recognition
Residual Soil	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broken by hand
Completely Weathered/Altered	Original minerals of rock have been almost entirely decomposed to secondary minerals, although original fabric may be intact; material can be granulated by hand
Highly Weathered/Altered	More than half of the rock is decomposed; rock is weakened so that a minimum 2-inch-diameter sample can be broken readily by hand across rock fabric
Moderately Weathered/Altered	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 2-inch-diameter sample cannot be broken readily by hand across rock fabric
Slightly Weathered/Altered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh/Unweathered	Rock shows no discoloration, loss of strength, or other effect of weathering/alteration

ROCK STRENGTH

Description	Recognition
Extremely Weak Rock	Can be indented by thumbnail
Very Weak Rock	Can be peeled by pocket knife
Weak Rock	Can be peeled with difficulty by pocket knife
Moderately Strong Rock	Can be indented 5 mm with sharp end of pick
Strong Rock	Requires one hammer blow to fracture
Very Strong Rock	Requires many hammer blows to fracture
Extremely Strong Rock	Can only be chipped with hammer blows

ROCK SCRATCH HARDNESS

Description	Recognition
Soft	Applicable only to plastic material
Friable	Can be easily crumbled by hand or reduced to powder; too soft to cut with a pocket knife
Low Hardness	Can be gouged deeply or carved with a pocket knife
Moderately Hard	Can be readily scratched by knife blade; scratch leaves heavy trace of dust and is readily visible after powder has been blown away
Hard	Can be scratched with a pocket knife only with difficulty; scratch produces little powder; traces of knife steel may be visible
Very Hard	Cannot be scratched with a pocket knife; knife steel marks are left on surface

ROCK FRACTURING

Description	Recognition
Intensely Fractured	Fractures spaced less than 2 inches apart
Highly Fractured	Fractures spaced 2 inches to 1 foot apart
Moderately Fractured	Fractures spaced 1 foot to 3 feet apart
Slightly Fractured	Fractures spaced 3 feet to 10 feet apart
Massive	Fracture spacing greater than 10 feet

CAMANCHE EMBANKMENTS SAFETY REVIEW

Log of Boring/Piezometer MB-1

Date(s) Drilled	9/16/08 - 9/23/08	Logged By	R. Harlan	Checked By	J. Barneich
Drilling Method	Hollow-Stem Auger 0-48 ft, Rotary Wash 48-193 ft; HQ Core 193-220 ft	Drill Bit Size/Type	8-inch-OD auger; 4-inch blade (rotary) bit; HQ core bit	Total Depth of Borehole	220.0 feet
Drill Rig Type	Mobile B-80	Drilling Contractor	PC Exploration	Approximate Surface Elevation	263 feet MSL
Groundwater Level(s)	Not recorded	Sampling Method	3-inch Shelby tube, SPT, HQ core	Hammer Data	Automatic hammer; 140 lbs / 30-inch drop
Borehole Location	Main Dam crest, 11 ft U/S of D/S edge, approx. Station 314+87	Borehole Completion	Four vibrating wire piezometers grouted in bentonite-cement mixture; tips set at 207.3 ft (#1 in switch box), 170.3 ft (#2), 120.3 ft (#3), and 70.3 ft (#4)		

Elevation, feet	Depth, feet	Samples: Type	Samples: Number	Blows Per 6 inches	Recovery, inches	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
	0					Asphalt						Start drilling at 1250 on 9/16/08. Advancing with 8-in. hollow-stem augers and intermittent Shelby tube and SPT sampling to 48 ft.
260						**CREST / SLOPE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SAND and COBBLES (GW), moderate yellowish brown, dry to damp, cobbles to 8 inches						
	5					**CORE (ZONE 1)** SILT to SANDY SILT (ML), LEAN CLAY to SANDY LEAN CLAY (CL), and SILTY SAND (SM), mixed and layered, moderate and dark yellowish brown and grayish brown, damp to moist, nonplastic to medium plasticity fines, mostly fine-grained sand						Smooth drilling at 5 ft.
255												
	10	SH	SH-1		22							Max. down pressure of 1000 psi for SH-1.
						SANDY SILT (ML), dark grayish brown, damp, nonplastic fines, fine-grained sand						
250						CLAYEY SAND (SC), dark grayish brown, damp to locally moist, dense, fine-grained sand, low plasticity fines, occasional very thin clay seams						
	15	SPT	SPT-1	13 19 18	17		42	20	12			Use AWJ rods for SPT sampling to 41 ft; NWJ rods below.
245						SILTY, CLAYEY SAND (SC-SM), dark grayish brown, damp, dense, fine-grained sand, low plasticity fines; occurs as silty sand with occasional gray clay clumps						
	20	SPT	SPT-2	12 17 20	14		46	17	4			HD: 15%<5μ End for 9/16/08 at 1720. Resume drilling on 9/17/08 at 0740.
240						SILT with SAND (ML), moderate yellowish brown, damp, low plasticity fines, fine-grained sand						
	25	SH	SH-2		24							Max. down pressure of 1200 psi for SH-2.
235						SANDY LEAN CLAY (CL), moderate yellowish brown, damp, very stiff to hard, low plasticity fines, fine-grained sand, occasional orange iron oxide-stained coarse-grained sand to fine gravel (weathered rock fragments)						
	30	SPT	SPT-3	8	17							

Report: TGP SOILPIEZ+LAB; File: TGP_CAMANCHE.GPJ; 12/16/2008

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Figure A-2

Elevation, feet	Depth, feet	Sample Type	Sample Number	Blows Per 6 inches	Recovery, inches	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
	30	SPT	SPT-3	14 17	17	SANDY LEAN CLAY (CL), moderate yellowish brown, damp, low plasticity fines, fine-grained sand [Core-Zone 1] (continued)	56	24	8			
230						SANDY SILT (ML), moderate yellowish brown, damp, nonplastic fines, fine-grained sand						
	35	SH	SH-3		22							Max. down pressure of 2000 psi for SH-3. Hard push. Tube pulled off sampling head in hole; auger past tube and recover by wedging rods in top of tube.
225						Becomes moderate brown						
	40	SPT	SPT-4	10 8 12	16	SANDY LEAN CLAY (CL), moderate grayish brown, damp, very stiff, medium plasticity fines, fine-grained sand	68	30	12			
220						SANDY LEAN CLAY (CL) to CLAYEY SAND (SC), dark brown, damp, low plasticity fines, fine-grained sand						
	45	SH	SH-4		22		51	22	9	11.5	122	Max. down pressure of 2000 psi for SH-4. TX-CIU' (6.5 ksf) HD: 17%<5μ Advance hole to 48 ft, pull augers, and set 5-in. casing to 49 ft. Resume drilling with rotary wash, using Revert-based drill fluid, 4-in. blade bit.
215												
	50	SH	SH-5		12	SANDY LEAN CLAY (CL), moderate yellowish brown, moist, medium plasticity fines, fine-grained sand with some medium- to coarse-grained angular sand, occasional gravel		34	20	16.5		Max. down pressure of 2000 psi for SH-5. Tip of tube badly dented. End for 9/17/08 at 52.2 ft. Resume drilling on 9/18/08 at 0802.
210												
	55					LEAN CLAY WITH SAND (CL), moderate yellowish brown, moist, medium plasticity fines, fine-grained sand						Max. down pressure of 1700 psi for SH-6; 800 psi for all but last few inches of push.
205		SH	SH-6		22		78 79	30 40 30	14 23 16	21.6 19.0	105 104	TX-CIU' (11.5 ksf) TX-CIU' (7.9 ksf) HD: 45%<5μ
	60					SANDY SILTY CLAY (CL-ML), moderate brown to moderate grayish brown, moist, very stiff to hard, low plasticity fines, fine-grained sand						SPT using NWJ rods.
200		SPT	SPT-5	7 14 18	17		52	17	4			HD: 18%<5μ
	65											

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES AND OTHER TESTS
	65						SANDY LEAN CLAY (CL), dark yellowish brown, moist, low to medium plasticity fines, fine-grained sand [Core-Zone 1]							Full drill fluid return.
195			SH-7		14				25	11	12.8			Max. down pressure of 2000 psi for SH-7.
	70						SILT (ML), medium gray, moist, low plasticity							
190														Max. down pressure of 2000 psi for SH-8; 800 psi for all but last few inches of push.
	75		SH-8		22		SANDY LEAN CLAY (CL), grayish brown, moist, low to medium plasticity fines, fine-grained sand	65 57	29 24	14 10	14.3 14.1	118 118		TX-CIU' (13.0 ksf) TX-CIU' (8.6 ksf) HD: 23%<5μ
185														Max. down pressure of 1400 psi for SH-9; 800 psi for all but last few inches of push.
			SH-9		23		Becomes moderate brown, increasing sand	52	29 27	10 9	18.1 21.8	110		TX-CIU' (9.4 ksf)
	80													
180			SPT-6	15 17 25	18		Becomes dark brown to grayish brown, damp	53	22	17				
	85													
							CLAYEY SAND (SC), dark grayish brown, moist, fine- to medium-grained sand, low plasticity fines, trace coarse-grained sand and fine gravel							Max. down pressure of 1700 psi for SH-10; 1000 psi for all but last few inches.
175			SH-10		23			27 34	25 25	12 14	15.5 15.0	122 121		TX-CIU' (14.4 ksf) TX-CIU' (10.0 ksf) HD: 17%<5μ
	90						SILTY SAND (SM), light grayish brown (sand) and moderate yellowish brown (fines), damp to moist, very dense, fine- to medium-grained sand; occurs as layered sand and silt							
170			SPT-7	28 49 38	18			33						
	95													Down pressure of 1000-2000 psi for SH-11; tip dented by gravel (rig chatter 99.5-100 ft).
165							SANDY LEAN CLAY (CL), dark brown, moist, low plasticity fines, fine- to medium-grained sand, trace coarse-grained sand to fine gravel							
			SH-11		18		Gravelly zone	54 50	27 25 23	13 10 7	11.6 10.0 11.9	123 124		TX-CIU' (15.8 ksf) TX-CIU' (10.8 ksf) HD: 15%<5μ
	100													

Report: TGP SOILPIEZ+LAB; File: TGP_CAMANCHE.GPJ; 12/16/2008

Elevation, feet	Depth, feet	Samples: Type	Samples: Number	Samples: Blows Per 6 inches	Samples: Recovery, inches	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
						SANDY LEAN CLAY (CL), dark brown, moist, low plasticity fines, fine- to medium-grained sand, trace coarse-grained sand to fine gravel [Core-Zone 1] (continued)						Continued full fluid return.
						SILTY, CLAYEY SAND (SC-SM) to SANDY SILTY CLAY (CL-ML), dark brown, moist, very dense / hard, fine- to medium-grained sand, low plasticity fines						
160		SPT	SPT-8	33 42 36	7		50	21	4			
	105											
						Becomes very moist to wet						
						Gravelly zone						Minor drill chatter 107.5-108 ft.
155		SH	SH-12		13			21	5	16.6		Max. down pressure of 2000 psi for SH-12.
	110											
						SANDY SILT (ML), dark grayish brown and dark yellowish brown, damp, nonplastic fines, fine-grained sand						
150		SPT	SPT-9	21 30 43	12		57	NP	NP			HD: 17%<5μ
	115											
145						Becomes dark yellowish brown, moist						Max. down pressure of 2000 psi for SH-13; end of tube dented.
		SH	SH-13		11			NP	NP	16.3		End for 9/18/08 at 1715. Resume drilling on 9/19/08 at 0715. Clean hole and find no overnight caving.
	120											
						SANDY SILTY CLAY (CL-ML), moderate yellowish brown, moist, very stiff, low plasticity fines, fine-grained sand						
140		SPT	SPT-10	7 9 19	18		66	20	7			
	125											
						SILTY SAND (SM), dark yellowish brown, moist, fine-grained sand, nonplastic fines						Down pressure of 1200-2000 psi for SH-14; tip dented.
135		SH	SH-14		16		33 43	NP NP	NP NP	18.0 12.7	110 117	TX-CIU' (15.8 ksf) TX-CIU' (12.2 ksf) HD: 12%<5μ
	130											
						SANDY LEAN CLAY (CL), dark yellowish brown, moist, hard, low to medium plasticity fines, fine-grained sand						
130						Few gravel						Minor drill chatter at 133 ft.
		SPT	SPT-11	20 43 46	12	SILT with SAND (ML), dark brown, moist, nonplastic	55	28	12			HD: 18%<5μ
						Gravelly zone						Drill chatter at 134-134.5 ft.
	135											

Elevation, feet	Depth, feet	Sample Type	Sample Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES AND OTHER TESTS
							SANDY LEAN CLAY (CL), dark yellowish brown, moist, hard, low to medium plasticity fines, fine-grained sand [Core-Zone 1] (continued)							Continued full fluid return.
							SANDY SILT (ML), dark brown and dark grayish brown, moist, nonplastic fines, fine-grained sand							Down pressure of 1200-2000 psi for SH-15; refusal 1.1 ft into push.
125			SH-15		12			53	NP	NP	13.5	120		TX-CIU' (13.0 ksf) HD: 12%<5µ
	140													
							SILTY, CLAYEY SAND (SC-SM) to SANDY SILTY CLAY (CL-ML), dark grayish brown and moderate yellowish brown, moist, very dense / hard, fine-grained sand, low plasticity fines; occurs as silty sand and sandy lean clay in alternating layers 2-4 inches thick							
120			SPT-12	18 26 34	14			50	21	5				Shredded wood fragments in return at 143 ft.
	145													
115			SH-16		0		SILTY SAND with GRAVEL (SM), dark yellowish brown, moist, very dense, fine- to medium-grained sand, nonplastic fines, fine to coarse rounded gravel possibly to 2 inches; occasional silt or clay layers							Down pressure of 1400-2000 psi for SH-16; tube dented and no recovery. Rig chatter from 148-158 ft.
	150													
			SPT-13	23 34 59	12			35	NP	NP				HD: 8%<5µ
110														Rods bouncing from 152.2-152.4 ft.
	155													
			SH-17		2		Becomes grayish brown and moderate yellowish brown							Max. down pressure of 2000 psi for SH-17; refusal 0.6 ft into push. Tip dented; sample bagged. End for 9/19/08 at 1500. Resume on 9/22/08 at 0945. Smooth drilling to 160 ft. Rig chatter 160-161.5 ft.
105														
	160													
							SANDY SILTY CLAY (CL-ML), dark brown and dark yellowish brown, damp, low plasticity fines, fine-grained sand, trace medium- to coarse-grained sand, few fine gravel in scattered beds							
100			SPT-14	20 31 40	8			53	24	5				HD: 17%<5µ
														Rig chatter from 163-167.5 ft; smooth 167.5-168 ft.
	165													
							CLAYEY SAND (SC), dark brown, damp to moist, fine-grained sand, low plasticity fines							
95			SH-18		6				27	8	14.3			Down pressure of 1500-2000 psi for SH-18; refusal 0.8 ft into push.
	170													

Elevation, feet	Depth, feet	Samples: Type / Number	Blows Per 6 inches	Recovery, inches	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
					CLAYEY SAND (SC), dark brown, damp to moist, fine-grained sand, low plasticity fines [Core-Zone 1] (continued)						Drilling slower below 169 ft to prevent pump pressure from increasing.
90					SANDY LEAN CLAY (CL), dark gray and dark grayish brown, damp to moist, hard, low plasticity fines, fine-grained sand						
		SPT-15	14 30 41	16		61	26	8			
	175										Down pressure of 1200-2000 psi for SH-19; refusal 1.2 ft into push.
85					SANDY SILTY CLAY (CL-ML), moderate yellowish brown, moist, low plasticity fines, fine-grained sand						
		SH-19		10		58	22 25	7 7	13.3 12.4	117	TX-CIU' (15.8 ksf) HD: 17%<5μ
	180										
					MEHRTEN FORMATION						
80		SPT-16	52	6	SANDY SILTSTONE, pale brown with orange iron oxide staining, moderately weathered, weak, low hardness, some fracturing, some iron oxide-stained surfaces	59					HD: 10%<5μ SPT refusal; rods bouncing at 183.5 ft.
	185				Becomes slightly weathered, unfractured (massive)						
					SILTY SANDSTONE, pale brown, slightly weathered, weak, low hardness, unfractured						
75		SPT-17	21 19 32	18		44					HD: 14%<5μ
	190										End for 9/22/08. Resume drilling on 9/23/08 at 0830.
					SILTSTONE, pale brown, slightly weathered, weak, low hardness, unfractured						Harder drilling at 192 ft.
70		SPT-18	18 >50	10	Faint iron oxide staining, some quartz grains	78					HD: 10%<5μ SPT refusal; rods bouncing at 193.8 ft. Switch to HQ coring; Run 1 cored over SPT-18 interval. Rec.: 100%; rock too weak for RQD.
		Run 1		14	Becomes fresh, subhorizontal layering						
	195										
65		Run 2		52	SANDSTONE, grayish brown, fine-grained, fresh, very weak, friable, wavy subhorizontal bedding at ~10°, occasional thin siltstone interbeds						Run 2: Rec.=86%; RQD=10% (100% for claystone at bottom of run, remainder too weak for RQD).
	200				CLAYSTONE, pale yellowish brown, fresh, moderately strong, low hardness						Continued full fluid return.
					SANDY SILTSTONE, pale brown, fresh, weak to moderately strong, low hardness						
60		Run 3		60	SILTY SANDSTONE, pale brown, slightly weathered, very weak to weak, friable to low hardness, fractured, some iron oxide-stained surfaces, some mechanical breakage						Run 3: Rec.=100%; RQD=50% (100% for first 2.5 ft, NA for rest of run).
	205				CLAYSTONE, mechanically broken						

Elevation, feet	Depth, feet	Type	Number	Blows Per 6 inches	Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES AND OTHER TESTS
							SANDY SILTSTONE, pale brown, fresh, weak, low hardness, unfractured							Total fluid loss at 205 ft; occasional return during run.
55			Run 4		40		CLAYSTONE, pale yellowish brown, slightly weathered, weak to moderately strong, low hardness, unfractured; iron oxide-stained breccia zone in core 207.6-207.9 ft							Run 4: Rec.=66%; RQD=30% (100% for claystone interval; NA for rest of run).
	210						SANDSTONE, grayish brown, fine-grained, fresh, very weak, friable, no cementation (core loss 208.4-214.5 ft)							Almost full fluid return below 210 ft.
50			Run 5		6									Run 5: Rec.=10%; RQD=0%.
45	215		Run 6		56		SANDY SILTSTONE / CLAYSTONE, pale yellowish brown and grayish brown, fresh, moderately strong, low hardness, unfractured, some coarse-grained sand-size angular volcanic rock fragments SANDY SILTSTONE, pale brown, very weak, friable Numerous mechanical breaks (tube packed)							Recovery last 0.5 ft of Run 5 in Run 6. Run 6: Rec.=94%; RQD=60% (100% for siltstone/claystone; NA for rest of run). Harder drilling from 216-218 ft. Continued near full fluid return.
	220						**Bottom of boring at 220.0 feet**							
40														
	225													
35														
	230													
30														
	235													
25														
	240													

CAMANCHE EMBANKMENTS SAFETY REVIEW

Log of Boring/Piezometer MB-2

Date(s) Drilled	9/29/08	Logged By	R. Harlan	Checked By	J. Barneich
Drilling Method	Hollow-Stem Auger 0-4.5 feet, Rotary Wash 4.5-21 feet	Drill Bit Size/Type	8-inch-OD auger; 4-inch blade bit	Total Depth of Borehole	21.0 feet
Drill Rig Type	Mobile B-80	Drilling Contractor	PC Exploration	Approximate Surface Elevation	110 feet MSL
Groundwater Level(s)	2.5 feet bgs ATD on 9/29/08	Sampling Method	SPT	Hammer Data	Automatic hammer; 140 lbs / 30-inch drop
Borehole Location	Main Dam, D/S toe, 10.5 ft R, 0.5 ft D/S of MD-11; approx. Sta. 313+00	Borehole Completion	Standpipe piezometer: 1-in.-dia. solid PVC 0-18.2 ft, porous 18.2-19.2 ft; Lapis Lustre #3 sand 17-21 ft, 1/4-in. bentonite pellets 15-17 ft, cement grout 0-15 ft		

Elevation, feet	Depth, feet	Samples: Type	Samples: Number	Samples: Blows Per 6 inches	Samples: Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES AND OTHER TESTS
110	0						**DOWNSTREAM SLOPE / TOE PROTECTION (ZONE 6)** SILTY GRAVEL with SAND (GM), yellowish brown, dry, occasional cobbles to 8 inches in upper 2 feet Becomes wet (▽)							Start drilling at 1105. Drill with 10-in. auger to 1.5 ft, 8-in. auger 1.5-4.5 ft. Wet auger cuttings from about 2.5 ft. Use NWJ rods for SPT sampling.
105	5	SPT	SPT-1	11 / 12 / 11	18		**TAILINGS** POORLY GRADED SAND WITH SILT (SP-SM), moderate and dark yellowish brown, wet, medium dense, mostly fine-grained sand	10						Drive SPT-1, then switch to rotary wash drilling at 4.5 ft, using Revert-based drill fluid, 4-in. blade bit.
		SPT	SPT-2	<5 / 4 / <6	11		POORLY GRADED SAND (SP), grayish brown, wet, loose, fine-grained sand, minor medium- to coarse-grained sand, trace silt	4						
100	10	SPT	SPT-3	<5 / <4 / <4	7		POORLY GRADED SAND WITH SILT (SP-SM), dark brown, wet, loose, fine-grained sand, minor medium- to coarse-grained sand	6						
		SPT	SPT-4	<4 / <4 / 3	6		Thin lens of medium- to coarse-grained sand	10						
95	15	SPT	SPT-5	3 / <6 / 4	5		POORLY GRADED SAND WITH GRAVEL (SP), grayish brown, wet, loose, fine- to coarse-grained sand, trace silt Possible gravelly zone	4						Rig chatter from 16.5-16.8 ft.
		SPT	SPT-6	2 / <4 / <5	1		SANDY SILT (ML) and POORLY GRADED SAND (SP), grayish brown, wet, loose, nonplastic fines, fine-grained sand							Poor recovery in SPT-6; coarse sand sloughing into hole. Blows at top of SPT-7 suspect due to sloughing and caving.
90	20	SPT	SPT-7	1 / 2 / 14	8		**MEHRTEN FORMATION** SILTSTONE, medium gray, weak, low hardness	30	NP	NP				
							Bottom of boring at 21.0 feet							Set 3-ft-long, 8-in.-dia. protective steel surface casing and cement in place on 10/1/08.
85	25													
80	30													

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Figure A-3

CAMANCHE EMBANKMENTS SAFETY REVIEW

Log of Boring/Piezometer MB-3

Date(s) Drilled	**9/25/08**	Logged By	**R. Harlan**	Checked By	**J. Barneich**
Drilling Method	**Hollow-Stem Auger 0-4.5 feet, Rotary Wash 4.5-16 feet**	Drill Bit Size/Type	**8-inch-OD auger; 4-inch blade bit**	Total Depth of Borehole	**16.0 feet**
Drill Rig Type	**Mobile B-80**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**104.5 feet MSL**
Groundwater Level(s)	**3.5 feet bgs ATD on 9/25/08**	Sampling Method	**SPT**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Main Dam, D/S toe, 12 ft R, 4.5 ft U/S of MD-20; approx. Sta. 315+00**	Borehole Completion	**Standpipe piezometer: 1-in.-dia. solid PVC 0-12.2 ft, porous 12.2-13.2 ft; Lapis Lustre #3 sand 10.8-16 ft, 1/4-in. bentonite pellets 9-10.8 ft, grout 0-9 ft**		

Elevation, feet	Depth, feet	Samples: Type	Samples: Number	Samples: Blows Per 6 inches	Samples: Recovery, inches	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
	0					**DOWNSTREAM SLOPE / TOE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SILT, SAND, and COBBLES (GW-GM), yellowish brown, dry, some cobbles to 8 inches						Start drilling at 1050 with 8-in. augers. Slow, rough drilling to 3.5 ft; 80 min. to auger through cobbles.
100	5					**TAILINGS** POORLY GRADED SAND with SILT (SP-SM), dark yellowish brown, wet, very loose to loose, fine-grained sand						Switch to rotary wash drilling at 4.5 ft, using Revert-based drill fluid, 4-in. blade bit. Use NWJ rods for SPT sampling.
		SPT	SPT-1	<2 <2 <2	13		8					HD: 3%<5μ
		SPT	SPT-2	1 1 <3	6		10					HD: 3%<5μ
95	10					POORLY GRADED SAND (SP), dark yellowish brown, wet, loose to medium dense, fine-grained sand, minor medium- to coarse-grained sand, trace silt						
		SPT	SPT-3	<4 5 <6	13		5					HD: 1%<5μ
						SILTY SAND (SM), grayish brown, wet, loose, fine-grained sand						
		SPT	SPT-4	<3 3 <3	17	SANDY SILT (ML) and POORLY GRADED SAND (SP), dark gray to moderate yellowish brown, wet, loose, nonplastic fines, fine-grained sand	34					HD: 5%<5μ
90	15					**MEHRTEN FORMATION** SILTSTONE, pale brown with trace iron oxide staining, moderately weathered, weak, low hardness; some moderately hard rock fragments						Harder drilling at 14 ft.
		SPT	SPT-5	31 80	12		59					HD: 7%<5μ
						Bottom of boring at 16.0 feet						Drill fluid measured at 0.5 ft bgs 1/2 hr after drilling, before installing piezometer. Set 3-ft-long, 8-in.-dia. protective steel surface casing and cement in place on 10/1/08.
85	20											
80	25											
75	30											

CAMANCHE EMBANKMENTS SAFETY REVIEW	Log of Boring/Piezometer MB-3 VWP Sheet 1 of 1

Date(s) Drilled	10/1/08 - 10/2/08	Logged By	R. Harlan	Checked By	J. Barneich
Drilling Method	Hollow-Stem Auger 0-4.5 feet, Rotary Wash 4.5-16 feet	Drill Bit Size/Type	8-inch-OD auger; 4-inch blade bit	Total Depth of Borehole	16.0 feet
Drill Rig Type	Mobile B-80	Drilling Contractor	PC Exploration	Approximate Surface Elevation	104.5 feet MSL
Groundwater Level(s)	Not recorded	Sampling Method	No sampling performed	Hammer Data	Not applicable
Borehole Location	Main Dam, D/S toe, 8 ft R of MB-3	Borehole Completion	Vibrating wire piezometer grouted in bentonite-cement mixture; tip set at 13 ft		

Columns: Elevation, feet | Depth, feet | SAMPLES (Type, Number, Blows Per 6 inches, Recovery, inches) | Graphic Log | MATERIAL DESCRIPTION | % Fines (<#200) | Liquid Limit | Plasticity Index | Moisture Content, % | Dry Unit Weight, pcf | Piezometer Installation Schematic | FIELD NOTES AND OTHER TESTS

Depth, feet	Elevation, feet	MATERIAL DESCRIPTION	FIELD NOTES AND OTHER TESTS
0		**DOWNSTREAM SLOPE / TOE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SILT, SAND, and COBBLES (GW-GM), yellowish brown, dry	Start drilling at 1250 on 10/1/08. Drill with 10-in. auger to 1.5 ft, 8-in. auger 1.5-4.5 ft.
		TAILINGS POORLY GRADED SAND with SILT (SP-SM), dark yellowish brown, wet	
5	100		Switch to rotary wash drilling at 4.5 ft, using Revert-based drill fluid, 4-in. blade bit. End for 10/1/08 at 1440. Resume drilling on 10/2/08 at 0955.
		POORLY GRADED SAND (SP), dark yellowish brown, wet	
10	95		
		SILTY SAND (SM), grayish brown, wet	
		SANDY SILT (ML) and POORLY GRADED SAND (SP), dark gray, wet	
		MEHRTEN FORMATION SILTSTONE, pale brown	
15	90		
16		**Bottom of boring at 16.0 feet**	Install vibrating wire piezometer, grouted in place, with tip at 13 ft on 10/2/08.
20	85		
25	80		
30	75		

CAMANCHE EMBANKMENTS SAFETY REVIEW

Log of Boring/Piezometer MB-4
Sheet 1 of 1

Date(s) Drilled	**9/30/08**	Logged By	**R. Harlan**	Checked By	**J. Barneich**
Drilling Method	**Hollow-Stem Auger 0-9 feet, Rotary Wash 9-23.5 feet**	Drill Bit Size/Type	**8-inch-OD auger; 4-inch blade bit**	Total Depth of Borehole	**23.5 feet**
Drill Rig Type	**Mobile B-80**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**110 feet MSL**
Groundwater Level(s)	**4 feet bgs ATD on 9/30/08**	Sampling Method	**3-inch Shelby tube, SPT**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Main Dam, D/S toe, 32.5 ft L, 3 ft D/S of MD-13; approx. Sta. _____**	Borehole Completion	**Standpipe piezometer: 1-in.-dia. solid PVC 0-20.2 ft, porous 20.2-21.2 ft; caved material and #3 sand 12-23.5 ft, 1/4-in. bentonite pellets 10-12 ft, grout 0-10 ft**		

Elevation, feet	Depth, feet	Samples: Type / Number	Blows Per 6 inches	Recovery, inches	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
110	0				**DOWNSTREAM SLOPE / TOE PROTECTION (ZONE 6)** SILTY GRAVEL with SAND and COBBLES (GM), yellowish brown, dry; Becomes moist, with numerous cobbles; Becomes wet ▽						Start drilling at 0750. Drill with 10-in. auger to 1.2 ft, 8-in. auger 1.2-9 ft. Rough, bouncing drilling 2-7 ft; 1 hr 40 min. to reach 7 ft. Water in augers at 4 ft.
105	5										
					TAILINGS POORLY GRADED SAND with SILT (SP-SM), dark grayish brown, wet, very loose to loose, fine-grained sand						Switch to rotary wash drilling at 9 ft, using Revert-based drill fluid, 4-in. blade bit. Using NWJ rods for SPT sampling.
100	10	SPT-1	<2 2 2	8	1/2-inch-dia. root or wood fragment	6					
					SANDY SILT (ML), dark grayish brown, wet, low plasticity fines, fine-grained sand						
		SH-1		0	FAT CLAY (CH), dark grayish brown, wet, very soft, high plasticity, moderately organic						Very low down pressure to push SH-1; no recovery.
95	15	SPT-2	0 0 0 <1	24		98	52	25			Rods settled 2 ft to 16 ft while setting hammer for SPT-2; one blow to 16.3 ft.
					SILT (ML), dark gray, wet, very soft, low plasticity						
		SH-2		24							SH-2 settled 6 in., very low down pressure for 12 in., 300 psi last 6 in.
					SILTY SAND (SM), dark gray, wet, very loose, fine-grained sand, nonplastic fines	17	NP	NP	34.8	89	HD: 3%<5μ
					Becomes dark grayish brown, with thin sandy silt seams, sand lenses, and scattered organics						
90	20	SPT-3	<1 <1 <1 <1	23		39	NP	NP			SPT advanced 17.5 in. with one blow, 5.5 in. with second blow.
		SPT-4	2 26 38	18	**MEHRTEN FORMATION** SILTSTONE, pale brown with orange iron oxide staining, moderately weathered, weak, low hardness	43	NP	NP			
					Bottom of boring at 23.5 feet						Set standpipe piezometer at end of drilling. Hole caves to 15 ft and, after sand added 14-15 ft, caves again to 12 ft. Set 3-ft-long, 8-in.-dia. protective steel surface casing and cement in place on 10/1/08.
85	25										
80	30										

Report: TGP SOILPIEZ+LAB; File: TGP_CAMANCHE.GPJ; 12/16/2008

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Figure A-6

CAMANCHE EMBANKMENTS SAFETY REVIEW	Log of Boring MB-5X Sheet 1 of 1

Date(s) Drilled	**9/30/08 - 10/1/08**	Logged By	**R. Harlan**	Checked By	**J. Barneich**
Drilling Method	**Hollow-Stem Auger**	Drill Bit Size/Type	**8-inch-OD auger**	Total Depth of Borehole	**8.7 feet**
Drill Rig Type	**Mobile B-80**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**115 feet MSL**
Groundwater Level(s)	**Not encountered**	Sampling Method	**No sampling performed**	Hammer Data	**Not applicable**
Borehole Location	**Main Dam, D/S toe, 9 ft R, 1 ft D/S of MD-5; approx. Sta. ______**	Borehole Completion	**Backfilled with cement grout to surface**		

Elevation, feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	SAMPLES: Blows Per 6 inches	SAMPLES: Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Gravel (>#4)	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	FIELD NOTES AND OTHER TESTS
115	0						**DOWNSTREAM SLOPE / TOE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SILT, SAND, and COBBLES (GW-GM), yellowish brown, dry Becomes damp							Start drilling at 1625 on 9/30/08. Drill with 10-in. auger to 1 ft, 8-in. auger 1-8.7 ft. Slow, rough drilling.
110	5													End for 9/30/08 at 4.5 ft at 1730. Resume drilling on 10/1/08 at 0745.
							Gravelly sand layer (easier drilling)							Easier, faster augering 7.5-8.5 ft. Very slow, rough drilling below 8.5 ft.
							Bottom of boring at 8.7 feet							Terminate hole at 8.7 ft after 1-1/2 hrs advancing from 8.5 ft; backfill with cement grout. Replace with Boring MB-5.
105	10													
100	15													
95	20													
90	25													
85	30													

Report: TGP SOIL+LAB; File: TGP_CAMANCHE.GPJ; 12/16/2008

TERRA / GeoPentech
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Figure A-7

CAMANCHE EMBANKMENTS SAFETY REVIEW

Log of Boring/Piezometer MB-5

Sheet 1 of 1

Date(s) Drilled	**10/2/08 - 10/3/08**	Logged By	**R. Harlan**	Checked By	**J. Barneich**
Drilling Method	**Air Percussion ODEX 0-19 feet, Rotary Wash 19-28.5 feet**	Drill Bit Size/Type	**5-inch-OD ODEX; 4-inch blade bit**	Total Depth of Borehole	**28.5 feet**
Drill Rig Type	**Mobile B-80**	Drilling Contractor	**PC Exploration**	Approximate Surface Elevation	**115 feet MSL**
Groundwater Level(s)	**8 feet bgs ATD on 10/2/08**	Sampling Method	**SPT**	Hammer Data	**Automatic hammer; 140 lbs / 30-inch drop**
Borehole Location	**Main Dam, D/S toe, 5 ft R, 1 ft U/S of MB-5X, 14 ft R of MD-5**	Borehole Completion	**Standpipe piezometer: 1-in.-dia. solid PVC 0-24.7 ft, porous 24.7-25.7 ft; Lapis Lustre #3 sand 23-28.5 ft, 1/4-in. bentonite pellets 21-23 ft, grout 0-21 ft**		

Elevation, feet	Depth, feet	Samples: Type	Samples: Number	Samples: Blows Per 6 inches	Samples: Recovery, inches	Graphic Log	MATERIAL DESCRIPTION	% Fines (<#200)	Liquid Limit	Plasticity Index	Moisture Content, %	Dry Unit Weight, pcf	Piezometer Installation Schematic	FIELD NOTES AND OTHER TESTS
115	0						**DOWNSTREAM SLOPE / TOE PROTECTION (ZONE 6)** WELL-GRADED GRAVEL with SILT, SAND, and COBBLES (GW-GM), yellowish brown, dry to damp							Start drilling with 5-in. ODEX at 1455 on 10/2/08. ODEX to 19 ft. Moist cuttings.
							Becomes moist							Fines on rock chips below 4 ft.
110	5													
							Becomes wet ∇							Blowing water and cuttings from hole below 8 ft.
105	10													
							Becomes looser (based on ODEX penetration rate)							Gradually easing advance, but continued rock chips below 14 ft.
100	15													
							TAILINGS SILTY SAND (SM), dark grayish brown, wet, very loose to loose, fine-grained sand; occasional thin layers of low plasticity silt, clean sand seams							Fast advance, fine sand blows from hole below 17 ft. Set 5 in. casing to 18.5 ft.
95	20													Switch to rotary wash drilling at 19 ft, using Revert-based drill fluid, 4-in. blade bit. Use NWJ rods for SPT sampling. Losing drill fluid. End for 10/2/08 after driving SPT to 22.7 ft. Resume drilling on 10/3/08 at 0745.
			SPT-1	1 <2 <2	20		SILT (ML), grayish brown, wet, soft, 1-inch layer; SILT (ML), grayish brown, wet, soft, 2-inch layer	34	NP	NP				
			SPT-2	<3 <4 <3	19		Several rounded gravel 1/2 inch to 1-1/2 inches dia. Two 1/2-inch lenses of SAND (SP), fine-grained	13	NP	NP				
90	25						Possible gravelly layer (gravel caving into hole)							Some rig chatter (gravel) at 25 ft; still losing fluid. Hole caves to 19 ft. Redrill to 26 ft for SPT-3. Higher blow counts 25.6-26 ft in gravel or rock chips, discarded.
			SPT-3	3 10	2			42	NP	NP				
							MEHRTEN FORMATION SILTSTONE, pale brown, weak, low hardness							
							Bottom of boring at 28.5 feet							Install standpipe piezometer at end of drilling.
85	30													

Report: TGP SOILPIEZ+LAB; File: TGP_CAMANCHE.GPJ; 12/16/2008

TERRA / GeoPentech
a Joint Venture

Enclosure 1

October 3, 2008

Robert C. Kirby,
Terra Engineers, Inc.
350 Sansome Street, Suite 830
San Francisco, CA 94104
Phone 888-888-4730 Fax 888-888-4731
BobKirby@terraengineers.com

Re: Standard Penetration Energy Measurements
Automatic Hammer on PC Exploration Drilling Drill Rig
Lake Comanche – Phase 2
Ione, California

Dear Mr. Kirby,

This report offers results of energy measurements and related calculations made on September 25, 2008 during Standard Penetration Testing (SPT) on a PC Exploration drill rig. Dynamic tests were performed on an instrumented section of NWJ drill rod attached to the sampler rod string. All dynamic measurements were obtained and recorded using a Pile Driving Analyzer®.

Equipment:

SPT energy measurements were made on SPT samplers and Mod. Cal. samplers driven by a 140# un-mounted auto-hammer/anvil system, deployed by a PC Exploration Mobile B-80 drill rig on September 25, 2008. The rig was tested while completing a boring at the toe of Comanche Dam. In total, 5 energy measurements were collected corresponding to 5 different samples at increasing depth in 1 boring.

Gregg Drilling and Testing, Inc. used a Model PAK Pile Driving Analyzer (PDA) to acquire and process measurements of force and velocity with impacts of the 140# auto-hammer on the sample rods. Two strain gauges mounted on a two foot section of NWJ rod measured force, while two piezoresistive accelerometers bolted on the same rod measured acceleration. The gauges were mounted approximately 6" from the top of the rod.

Analog signals from the gauges and accelerometers were collected, digitized, displayed in real-time, and stored by the PDA. Selected output from the PDA for each recorded impact of the hammer included:

- Maximum force in the rod (FMX)
- Maximum velocity in the rod (VMX)
- Maximum calculated transferred energy (EMX)
- Blows per minute (BPM)
- Energy transferred to the rods in the time 2L/c (E2E)

Data and Calculations:

The purpose of testing was to measure the energy transferred from the hammer to the drill rod and to calculate the energy efficiency of the hammer. The PDA measurements of force and velocity were reviewed after field testing and analyzed to calculate the transferred energy (EMX).

The maximum energy transferred past the gauge location, EMX, is computed by the PDA using force (F) and velocity (V) records as follows:

$$EMX = \int_a^b F(t)V(t)dt$$

The time "a" corresponds to the start of the record when the energy transfer begins and "b" is the time at which energy transferred to the rod reaches a maximum value. The energy transferred in the time 2L/c (i.e. the time for the first compression wave to travel to the SPT sampler and back), is defined as E2E, and is usually used to define the efficiency of the hammer/anvil system.

Results:

Table 1 summarizes the average calculated energies for each sample tested as well as the type of sample and depth. It is shown that the overall average (E2E) energy for this system is 65.1%. Appendix A provides plots and tables of PDA results for all hammer blows at each sampling depth. The plots and tables present selected measured and calculated results as a function of blow number. The results include:

- the blow number
- depth
- BLC (blow count in blows per foot)
- FMX (maximum rod force)
- VMX (maximum rod velocity)
- EMX (maximum transferred energy)
- BPM (blows per minute)
- E2E (energy transferred in time 2L/c)

At the end of each table is a statistical evaluation of the results for each variable. The tables also include the average energy, standard deviation, maximum and minimum measured energies.

If you have any questions or comments on this report, please do not hesitate to call our office at (925) 313-5800.

Sincerely,

Virgil A. Baker
Geotechnical Manager

Client: Terra Engineers, Inc.
Project: Comanche Dam - Phase 2
Date: 25 September 2008
Boring: MB-3

TABLE 1 SPT Eneryg Summary

Sample #	Sampler	Length of Sample Rod (ft)	Sampler Length (ft)	Total Rod Length* (ft)	Depth of Sample (Below Grade)	Total Blows Analyzed by PDA	Average Energy Transferred to Rods (% of Theoretical Max.)	Maximum Efficiency Recorded (%)	Mminimum Efficiency Recorded (%)	Standard Deviation
MB-3@6.00	SPT	6.40	3.67	11.57	6.00	3	62	63.8	61	1.3
MB-3@7.5	SPT	7.80	4.17	13.47	7.50	5	68.6	69.3	68.1	0.5
MB-3@10.0	SPT	10.70	4.17	16.37	10.00	12	59.0	61.9	55.7	1.5
MB-3@12.5	SPT	12.90	4.17	18.57	12.50	8	64.9	65.7	62.8	1.0
MB-3@15.0	SPT	15.00	4.17	20.67	15.00	92	71.2	75.2	56.6	3.5
						Average:	65.1			

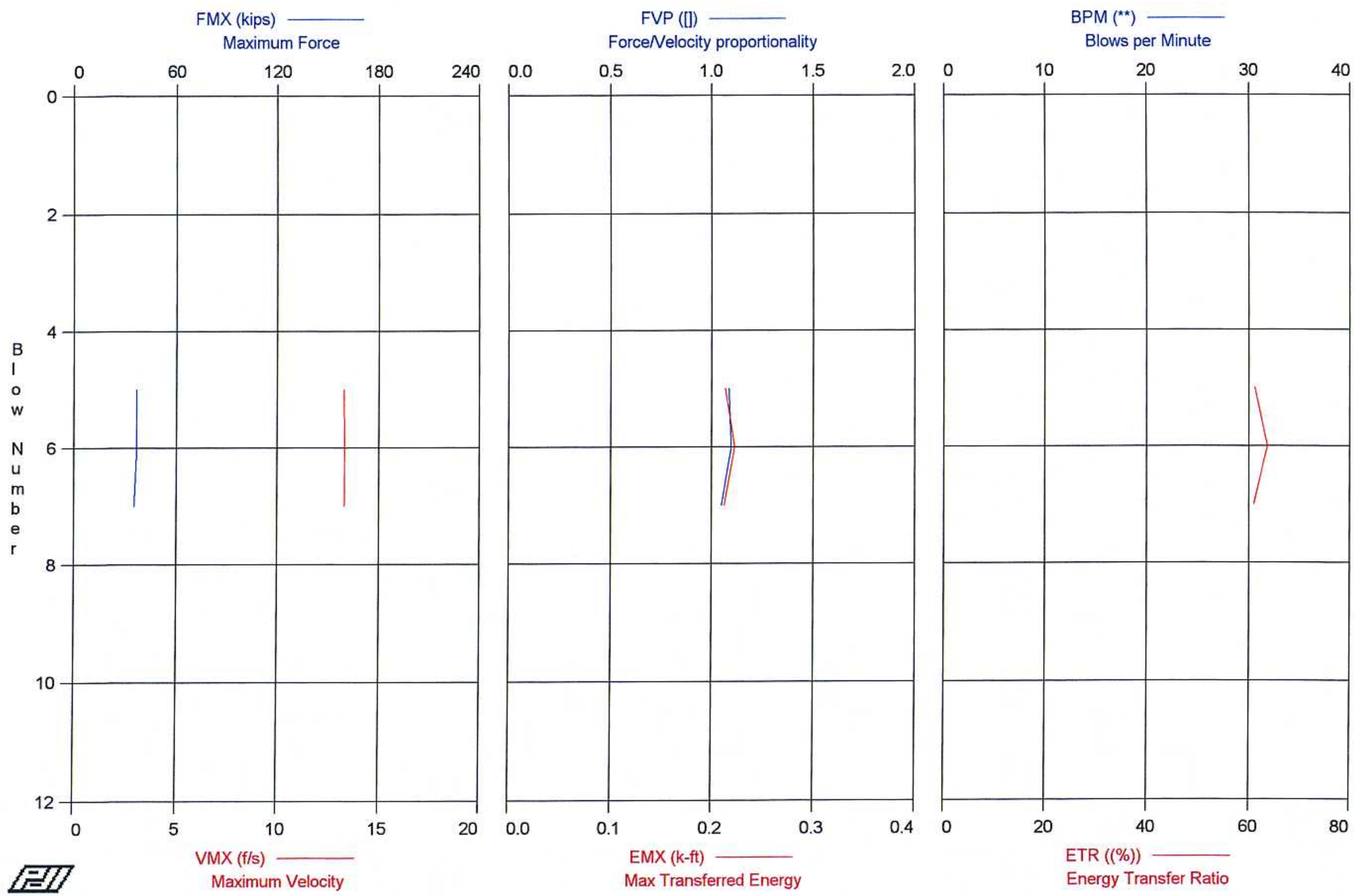
* Total rod length including sampler, rod adaptors, and instrumented rod section below gauges

PDIPLOT Ver. 2008.2 - Printed: 2-Oct-2008

Gregg In Situ - Case Method Results

Test date: 25-Sep-2008

Comanche Dam - Phase 2 - MB-3@6.00



Gregg In Situ
Case Method Results

PDIPLOT Ver. 2008.2 - Printed: 2-Oct-2008

Comanche Dam - Phase 2 - MB-3@6.00
OP: VAB

140lb AUTO HAMMER
Test date: 25-Sep-2008

AR: 1.39 in^2
LE: 11.57 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.75

FMX: Maximum Force
VMX: Maximum Velocity
EMX: Max Transferred Energy

BPM: Blows per Minute
ETR: Energy Transfer Ratio

BL#	FMX	VMX	EMX	BPM	ETR
	kips	f/s	k-ft	**	(%)
5	36	13.3	0.2	0.0	61.2
6	36	13.3	0.2	0.0	63.8
7	35	13.3	0.2	0.0	61.0
Average	36	13.3	0.2	**	62.0
Std. Dev.	1	0.0	0.0	**	1.3
Maximum	36	13.3	0.2	**	63.8
@ Blow#	6	6	6	**	6
Minimum	35	13.3	0.2	**	61.0

Total number of blows analyzed: 3

Time Summary

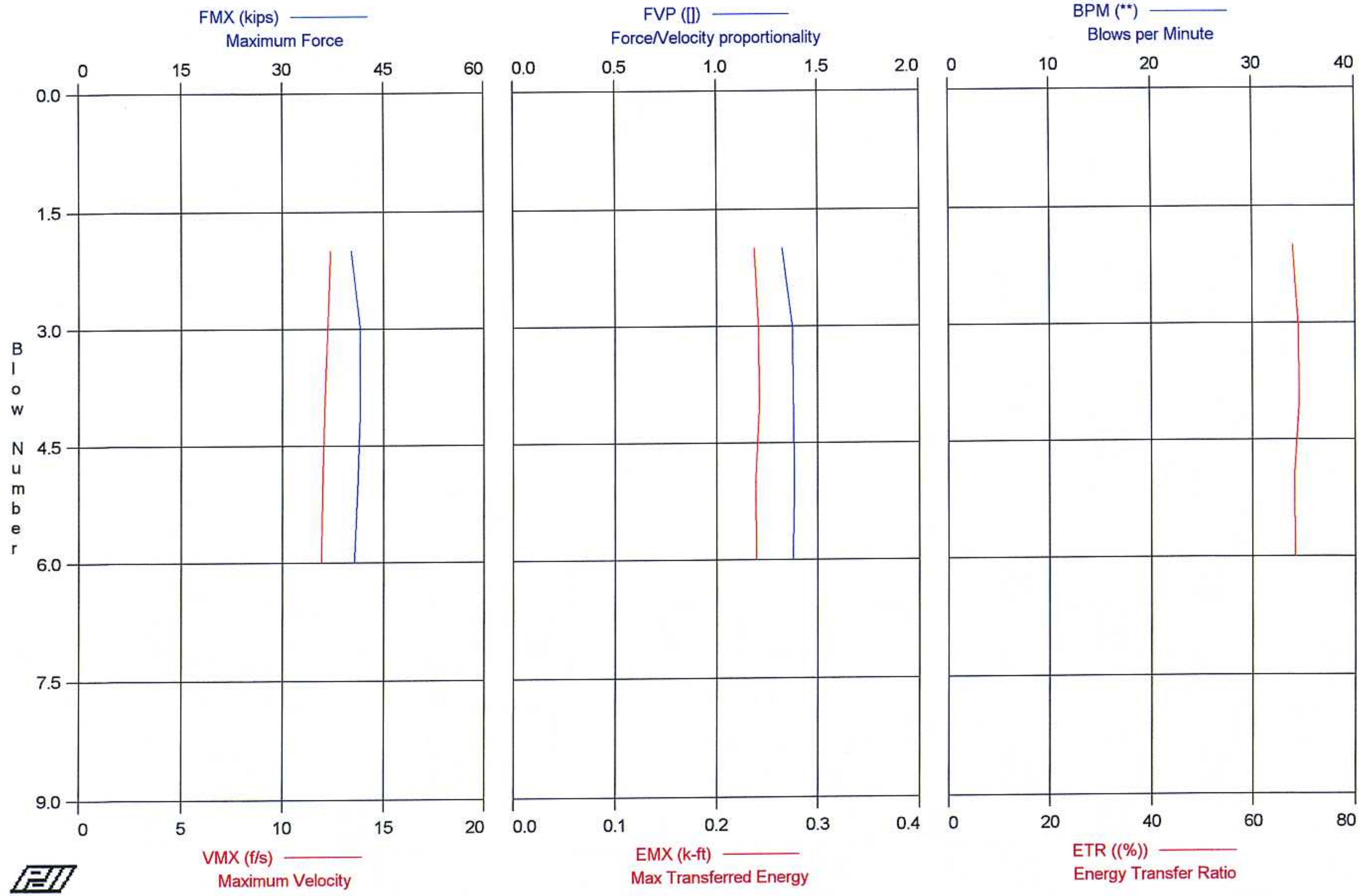
Drive 49 seconds 1:20:16 PM - 1:21:05 PM (9/25/2008) BN 1 - 7

PDIPLOT Ver. 2008.2 - Printed: 2-Oct-2008

Gregg In Situ - Case Method Results

Comanche Dam - Phase 2 - MB-3@7_50

Test date: 25-Sep-2008



Gregg In Situ
Case Method Results

PDIPLOT Ver. 2008.2 - Printed: 2-Oct-2008

Comanche Dam - Phase 2 - MB-3@7_50
OP: VAB

140lb AUTO HAMMER
Test date: 25-Sep-2008

AR: 1.39 in^2
LE: 13.47 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.75

FMX: Maximum Force
VMX: Maximum Velocity
EMX: Max Transferred Energy

BPM: Blows per Minute
ETR: Energy Transfer Ratio

BL#	FMX kips	VMX f/s	EMX k-ft	BPM **	ETR (%)
2	40	12.4	0.2	0.0	68.1
3	41	12.2	0.2	0.0	69.1
4	41	12.1	0.2	0.0	69.3
5	41	12.0	0.2	0.0	68.3
6	41	11.9	0.2	0.0	68.5
Average	41	12.1	0.2	**	68.6
Std. Dev.	1	0.2	0.0	**	0.5
Maximum	41	12.4	0.2	**	69.3
@ Blow#	4	2	4	**	4
Minimum	40	11.9	0.2	**	68.1

Total number of blows analyzed: 5

Time Summary

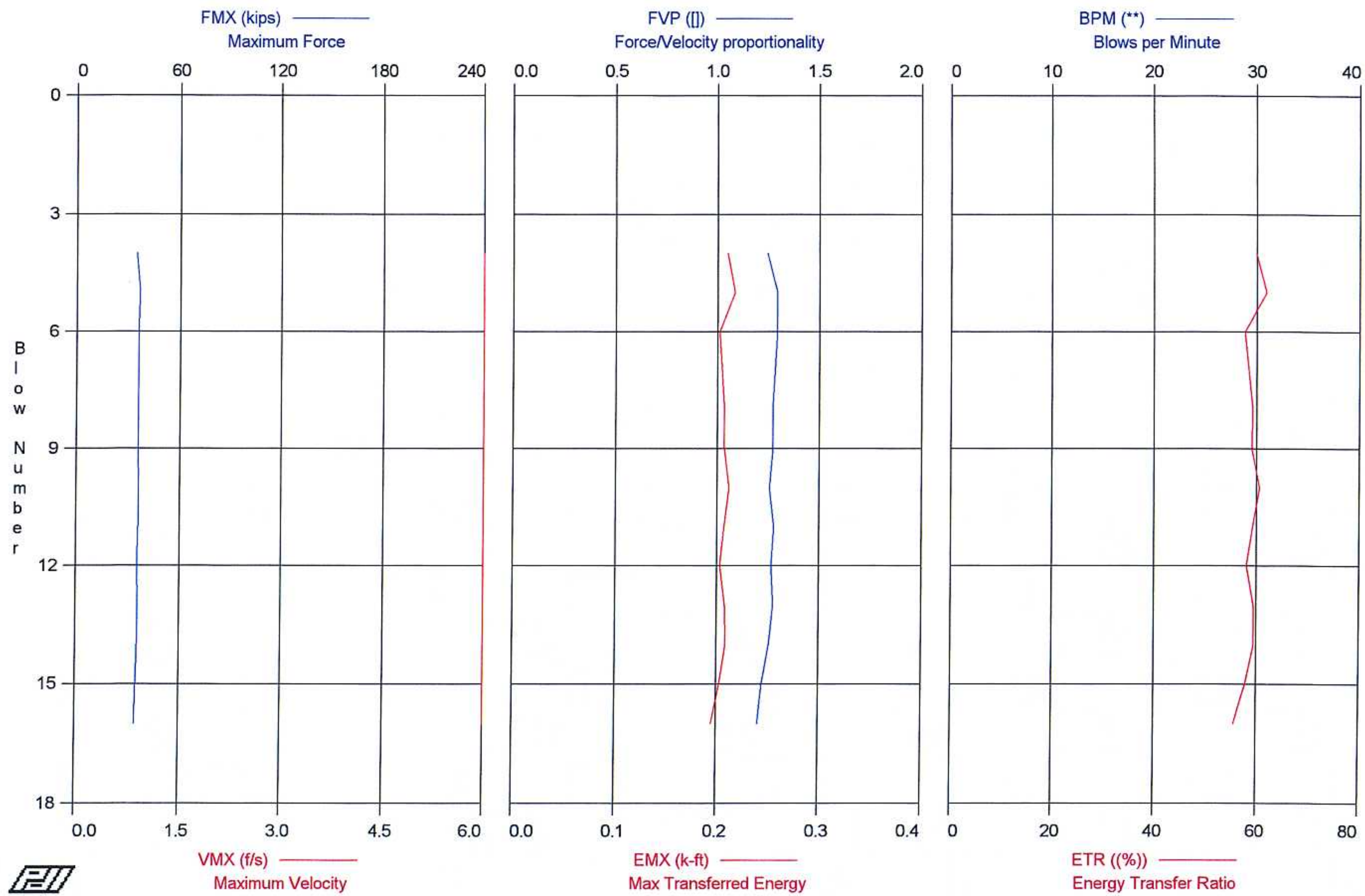
Drive 58 seconds 1:51:27 PM - 1:52:25 PM (9/25/2008) BN 1 - 6

PDIPLOT Ver. 2008.2 - Printed: 2-Oct-2008

Gregg In Situ - Case Method Results

Test date: 25-Sep-2008

Comanche Dam - Phase 2 - MB-3@10_00



Gregg In Situ
Case Method Results

PDIPLOT Ver. 2008.2 - Printed: 2-Oct-2008

Comanche Dam - Phase 2 - MB-3@10_00
OP: VAB

140lb AUTO HAMMER
Test date: 25-Sep-2008

AR: 1.39 in^2
LE: 16.37 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.75

FMX: Maximum Force
VMX: Maximum Velocity
EMX: Max Transferred Energy

BPM: Blows per Minute
ETR: Energy Transfer Ratio

BL#	FMX	VMX	EMX	BPM	ETR
	kips	f/s	k-ft	**	(%)
4	34	11.5	0.2	0.0	60.0
5	36	12.3	0.2	0.0	61.9
6	35	12.1	0.2	30.1	57.7
8	35	11.2	0.2	0.0	59.2
9	35	11.1	0.2	0.0	59.1
10	36	11.4	0.2	0.0	60.6
11	35	11.1	0.2	0.0	59.2
12	35	11.1	0.2	0.0	58.1
13	35	11.1	0.2	0.0	59.4
14	35	11.3	0.2	0.0	59.6
15	34	11.3	0.2	0.0	57.9
16	34	11.4	0.2	0.0	55.7
Average	35	11.4	0.2	30.1	59.0
Std. Dev.	1	0.4	0.0	0.0	1.5
Maximum	36	12.3	0.2	30.1	61.9
@ Blow#	5	5	5	6	5
Minimum	34	11.1	0.2	30.1	55.7

Total number of blows analyzed: 12

Time Summary

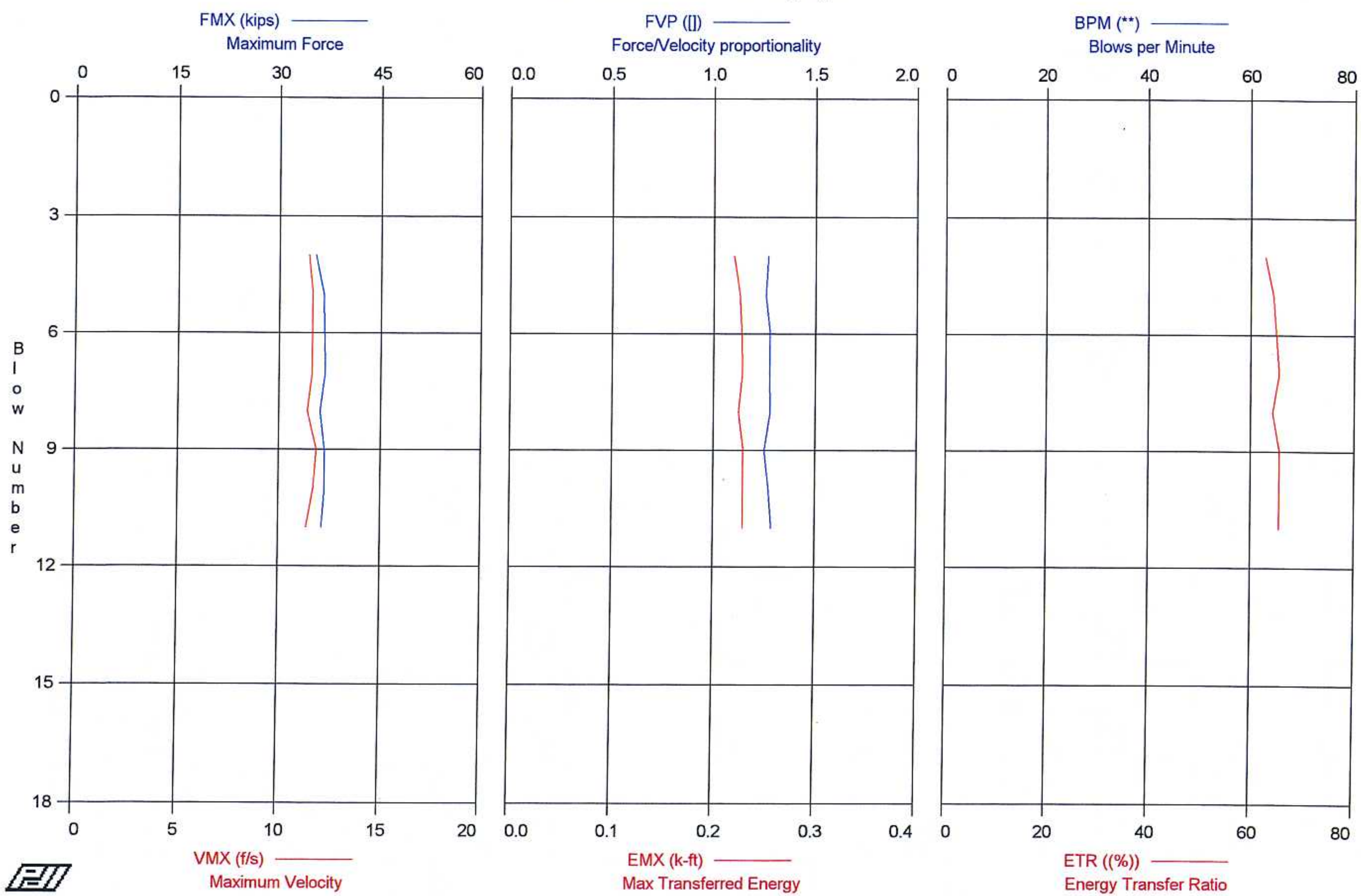
Drive 3 minutes 54 seconds 2:36:00 PM - 2:39:54 PM (9/25/2008) BN 1 - 16

PDIPLOT Ver. 2008.2 - Printed: 2-Oct-2008

Gregg In Situ - Case Method Results

Test date: 25-Sep-2008

Comanche Dam - Phase 2 - MB-3@12_50



Gregg In Situ
Case Method Results

PDIPLOT Ver. 2008.2 - Printed: 2-Oct-2008

Comanche Dam - Phase 2 - MB-3@12_50
OP: VAB

140lb AUTO HAMMER
Test date: 25-Sep-2008

AR: 1.39 in^2
LE: 18.57 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.75

FMX: Maximum Force
VMX: Maximum Velocity
EMX: Max Transferred Energy

BPM: Blows per Minute
ETR: Energy Transfer Ratio

BL#	FMX	VMX	EMX	BPM	ETR
	kips	f/s	k-ft	**	(%)
4	35	11.4	0.2	0.0	62.8
5	37	11.7	0.2	0.0	64.4
6	37	11.6	0.2	0.0	65.0
7	37	11.6	0.2	0.0	65.6
8	36	11.4	0.2	0.0	64.4
9	37	11.9	0.2	0.0	65.7
10	37	11.7	0.2	0.0	65.7
11	36	11.4	0.2	0.0	65.7
Average	36	11.6	0.2	**	64.9
Std. Dev.	0	0.2	0.0	**	1.0
Maximum	37	11.9	0.2	**	65.7
@ Blow#	10	9	9	**	10
Minimum	35	11.4	0.2	**	62.8

Total number of blows analyzed: 8

Time Summary

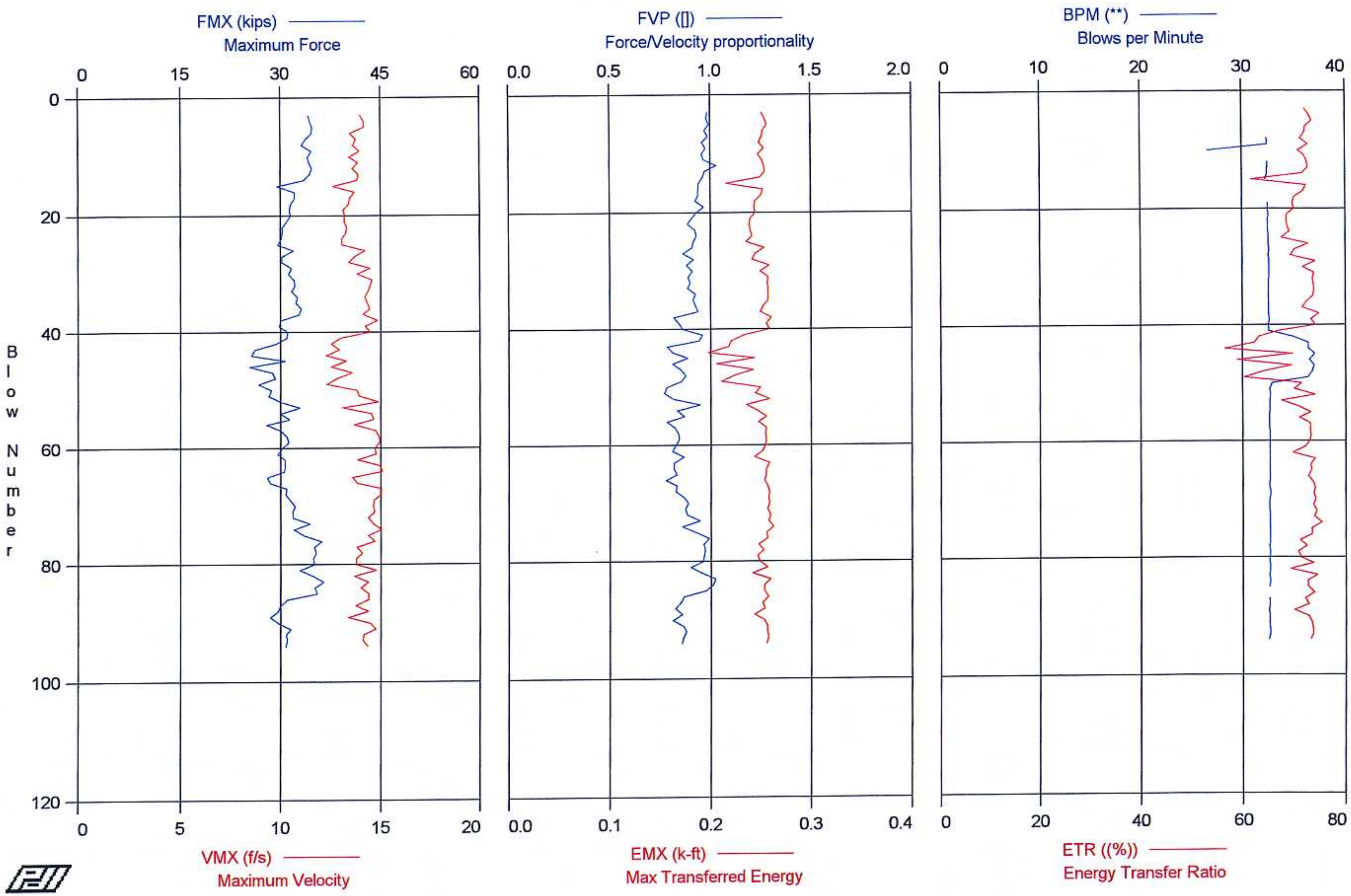
Drive 1 minute 16 seconds 3:01:16 PM - 3:02:32 PM (9/25/2008) BN 1 - 13

PDIPLOT Ver. 2008.2 - Printed: 2-Oct-2008

Gregg In Situ - Case Method Results

Test date: 25-Sep-2008

Comanche Dam - Phase 2 - MB-3@15_00



Comanche Dam - Phase 2 - MB-3@15_00
OP: VAB

140lb AUTO HAMMER
Test date: 25-Sep-2008

AR: 1.39 in^2
LE: 20.67 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.75

FMX: Maximum Force
VMX: Maximum Velocity
EMX: Max Transferred Energy

BPM: Blows per Minute
ETR: Energy Transfer Ratio

BL#	FMX kips	VMX f/s	EMX k-ft	BPM **	ETR (%)
3	34	14.0	0.3	0.0	72.1
4	34	14.2	0.3	0.0	72.9
5	35	14.1	0.3	0.0	73.5
6	35	13.5	0.3	32.4	72.2
7	34	13.8	0.3	0.0	71.9
8	33	13.6	0.2	32.3	71.2
9	35	13.9	0.3	32.4	72.7
10	34	13.4	0.2	26.7	70.8
11	34	13.9	0.3	0.0	72.0
12	35	13.6	0.3	32.5	72.4
13	34	13.9	0.3	32.4	72.7
14	33	13.8	0.3	32.4	71.5
15	29	12.6	0.2	32.2	61.8
16	32	13.7	0.3	0.0	72.2
17	32	13.4	0.3	28.3	71.7
18	32	13.4	0.2	0.0	70.0
19	31	13.1	0.2	32.5	69.7
20	32	13.2	0.2	32.4	69.9
21	31	13.2	0.2	32.5	68.5
22	30	13.3	0.2	32.4	68.4
23	30	13.2	0.2	32.5	68.6
24	30	13.1	0.2	32.5	69.0
25	30	13.1	0.2	32.5	67.4
26	32	14.2	0.3	32.5	72.7
27	30	13.7	0.2	32.5	70.1
28	30	13.4	0.2	32.4	69.3
29	32	14.5	0.3	32.5	74.0
30	31	13.8	0.3	32.5	71.6
31	32	14.6	0.3	32.5	73.7
32	32	14.5	0.3	32.5	73.5
33	32	14.4	0.3	32.5	73.7
34	33	14.2	0.3	32.5	73.7
35	32	14.3	0.3	32.5	73.6
36	33	14.4	0.3	32.5	72.3
37	33	14.1	0.3	32.5	71.5
38	30	14.8	0.3	32.5	74.7
39	30	14.2	0.3	32.5	73.2
40	31	14.5	0.3	32.5	74.0
41	31	13.0	0.2	32.5	66.8
42	29	12.5	0.2	34.8	63.0
43	26	12.9	0.2	36.3	62.3
44	26	12.3	0.2	36.4	56.6
45	31	13.3	0.2	36.9	69.7
46	25	12.6	0.2	36.5	59.0
47	29	13.6	0.2	36.9	69.4
48	29	12.8	0.2	36.7	64.0
49	27	12.3	0.2	36.3	60.4
50	29	13.8	0.3	32.8	71.3
51	28	13.9	0.2	32.5	69.9
52	30	14.9	0.3	32.6	74.0
53	33	13.1	0.2	32.6	67.5
54	30	14.5	0.2	32.6	70.7
55	31	14.6	0.3	32.5	73.1
56	28	13.7	0.2	32.5	70.9
57	30	14.7	0.3	32.6	73.1
58	31	14.9	0.3	32.6	72.9

Comanche Dam - Phase 2 - MB-3@15_00
OP: VAB

140lb AUTO HAMMER
Test date: 25-Sep-2008

BL#	FMX	VMX	EMX	BPM	ETR
	kips	f/s	k-ft	**	(%)
59	31	14.9	0.3	32.6	73.2
60	30	14.7	0.3	32.6	72.9
61	30	14.8	0.3	32.6	72.0
62	31	13.8	0.2	32.6	69.6
63	31	15.0	0.3	32.6	73.9
64	31	15.1	0.3	32.6	73.0
65	28	13.6	0.3	32.6	73.3
66	28	13.8	0.3	32.6	72.6
67	31	15.0	0.3	32.6	73.5
68	31	15.0	0.3	32.6	73.9
69	31	14.6	0.3	32.6	73.6
70	32	14.6	0.3	32.6	74.1
71	32	14.7	0.3	32.5	73.5
72	32	14.4	0.3	32.6	74.2
73	34	14.6	0.3	32.6	73.8
74	32	15.0	0.3	32.5	75.2
75	33	14.4	0.3	32.6	73.2
76	36	14.7	0.3	32.5	73.4
77	35	13.8	0.2	32.6	71.0
78	35	14.1	0.3	32.5	72.3
79	35	13.8	0.2	32.6	70.6
80	35	13.7	0.2	32.5	71.1
81	33	14.7	0.3	32.5	73.6
82	35	13.7	0.2	32.6	69.1
83	36	14.4	0.3	32.5	74.3
84	35	14.0	0.3	32.6	72.5
85	36	14.4	0.3	32.5	72.6
86	31	14.4	0.3	0.0	73.8
87	30	13.7	0.3	32.5	72.0
88	30	14.4	0.3	32.5	72.7
89	28	13.4	0.2	32.5	69.8
90	30	14.5	0.3	32.5	72.7
91	31	14.7	0.3	32.5	73.3
92	31	14.1	0.3	32.5	73.5
93	31	14.1	0.3	32.6	73.6
94	31	14.3	0.3	32.5	73.1
Average	31	14.0	0.2	32.8	71.2
Std. Dev.	2	0.7	0.0	1.4	3.5
Maximum	36	15.1	0.3	36.9	75.2
@ Blow#	83	64	74	45	74
Minimum	25	12.3	0.2	26.7	56.6

Total number of blows analyzed: 92

Time Summary

Drive 3 minutes 48 seconds 3:23:06 PM - 3:26:54 PM (9/25/2008) BN 1 - 94

APPENDIX B

GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

September 23, 2008

Terra Engineers
Attn: Bob Kirby
350 Sansome St., Suite 830
San Francisco, California 94104

Subject: CPT Site Investigation
Camanche Reservoir
Acampo, California
GREGG Project Number: 08-240MA

Dear Mr. Kirby:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☒
4	Resistivity Cone Penetration Tests	(RCPTU)	☐
5	UVOST Laser Induced Fluorescence	(UVOST)	☐
6	Groundwater Sampling	(GWS)	☐
7	Soil Sampling	(SS)	☐
8	Vapor Sampling	(VS)	☐
9	Vane Shear Testing	(VST)	☐
10	SPT Energy Calibration	(SPTE)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (925) 313-5800.

Sincerely,
GREGG Drilling & Testing, Inc.

Mary Walden
Operations Manager

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (Feet)	Depth of Groundwater Samples (Feet)	Depth of Soil Samples (Feet)	Depth of Pore Pressure Dissipation Tests (Feet)
MC-01	9/18/08	190.5	-	-	24.9, 25.1, 50.0, 75.0, 100.1, 125.0, 150.1, 175.0, 190.6
MC-02	9/18/08	21	-	-	-
MC-03	9/17/08	19	-	-	-
MC-04	9/19/08	26	-	-	-
MC-06	9/16/08	188	-	-	25.1, 75.1, 125.0, 175.0
MC-07	9/15/08	191	-	-	25.8, 75.1, 125.0, 175.0
MC-08	9/16/08	182	-	-	25.1, 75.3, 125.0, 175.0
MC-09	9/17/08	162	-	-	25.1, 125.0, 162.2
MC-10	9/19/08	18	-	-	-
MC-11	9/17/08	18	-	-	-

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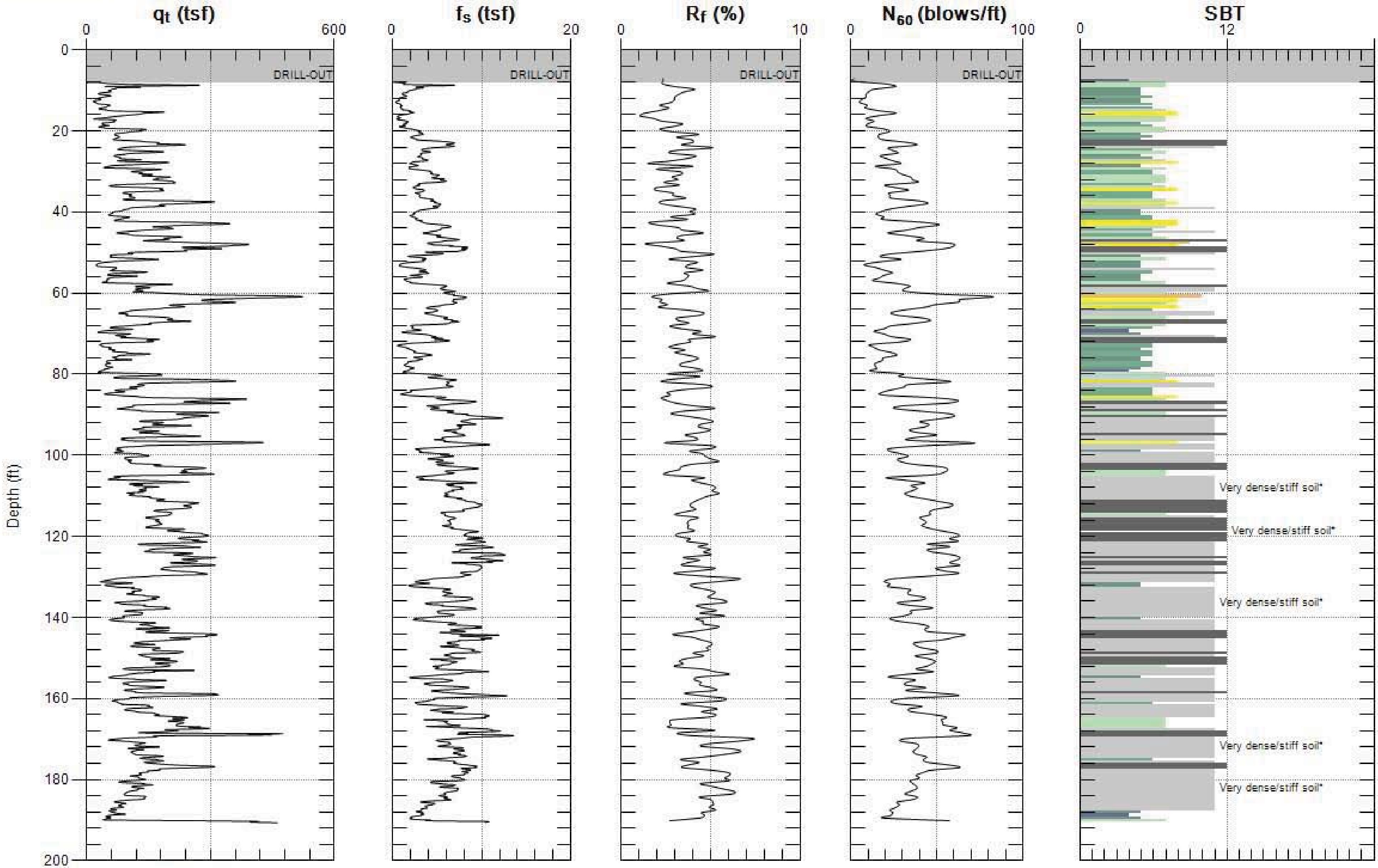
Copies of ASTM Standards are available through www.astm.org

GREGG TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-1 Date: 9/18/2008 09:03



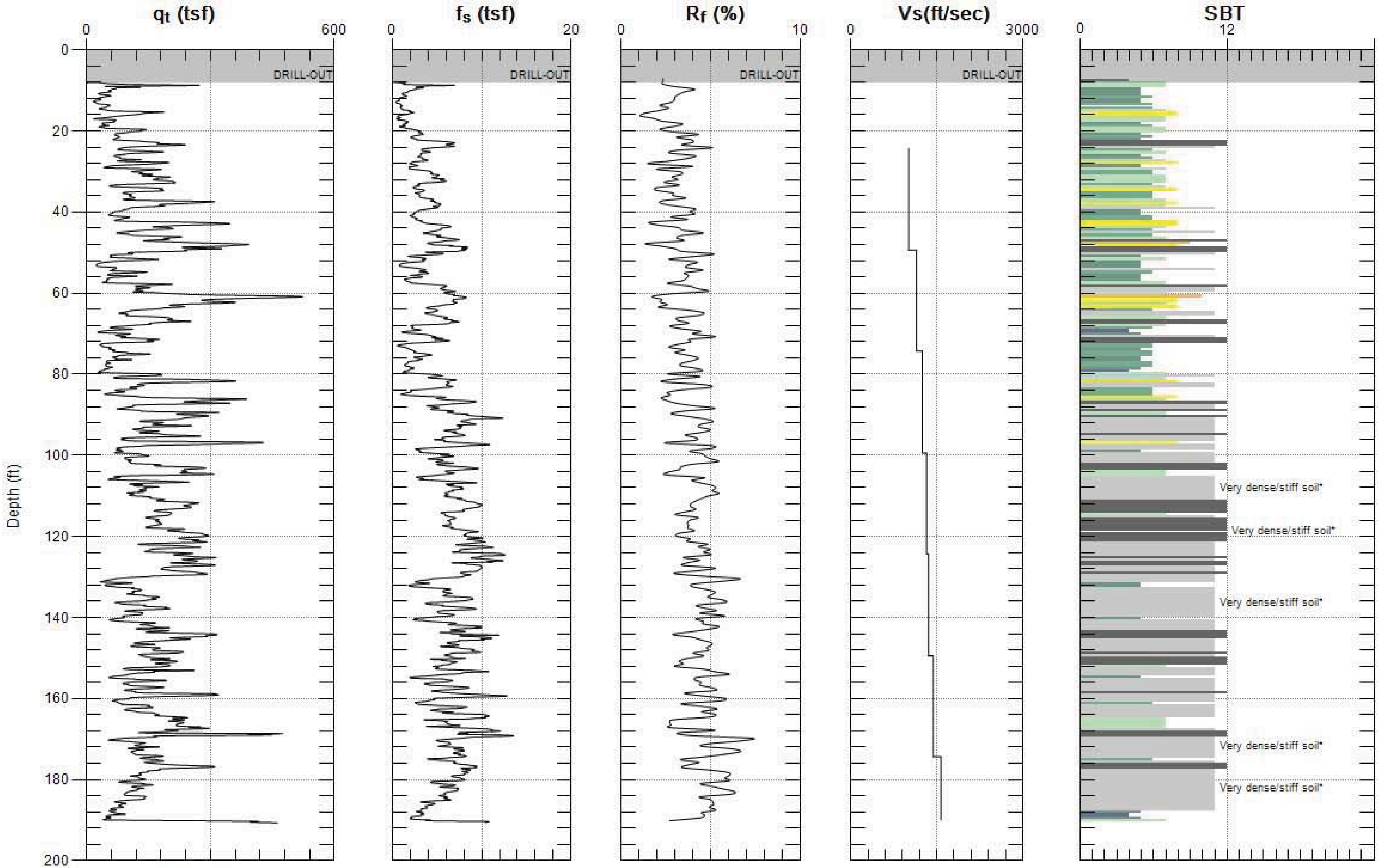
Max. Depth: 190.617 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-1 Date: 9/18/2008 09:03



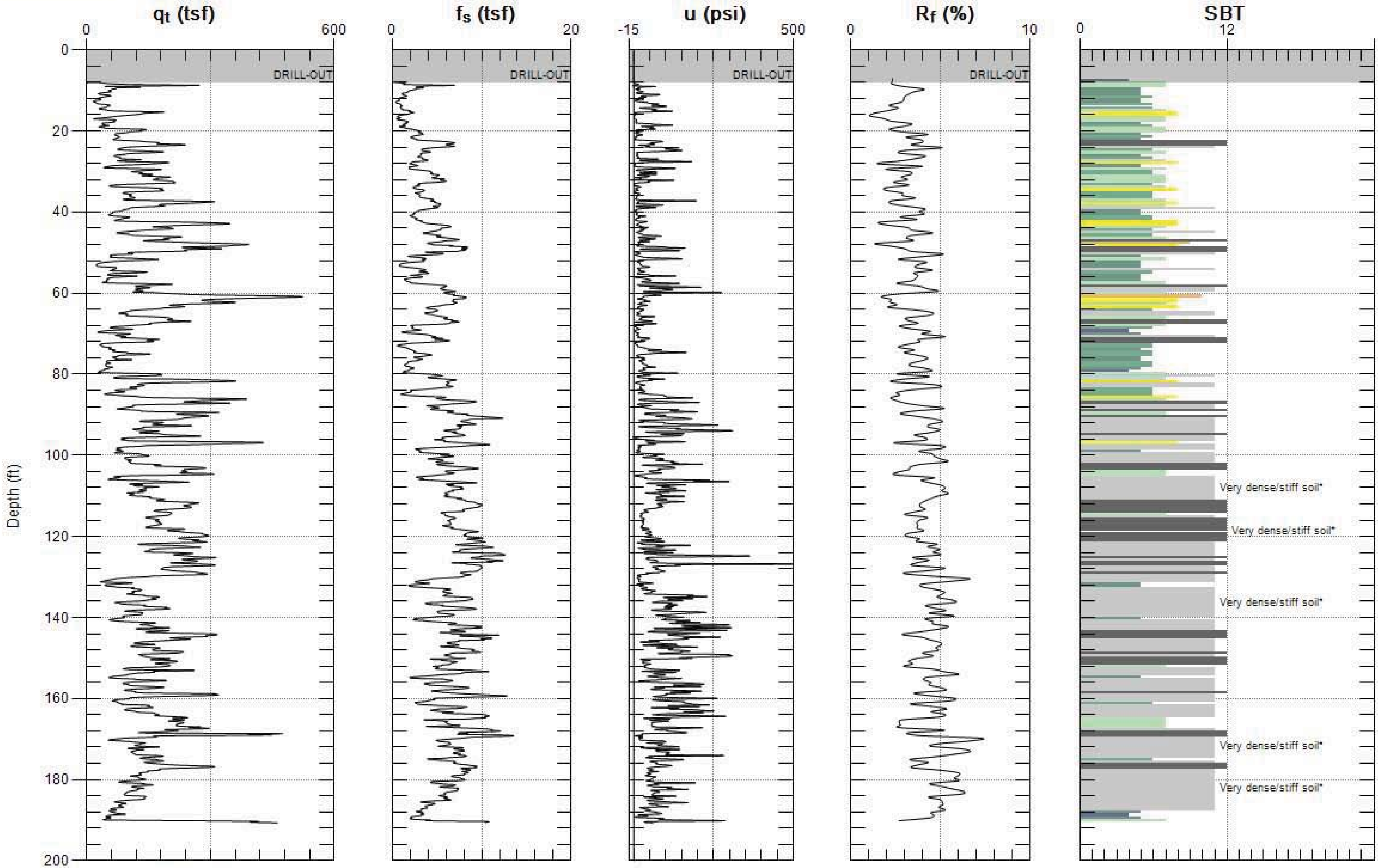
Max. Depth: 190.617 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TERRA ENGINEERS**

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-1 Date: 9/18/2008 09:03



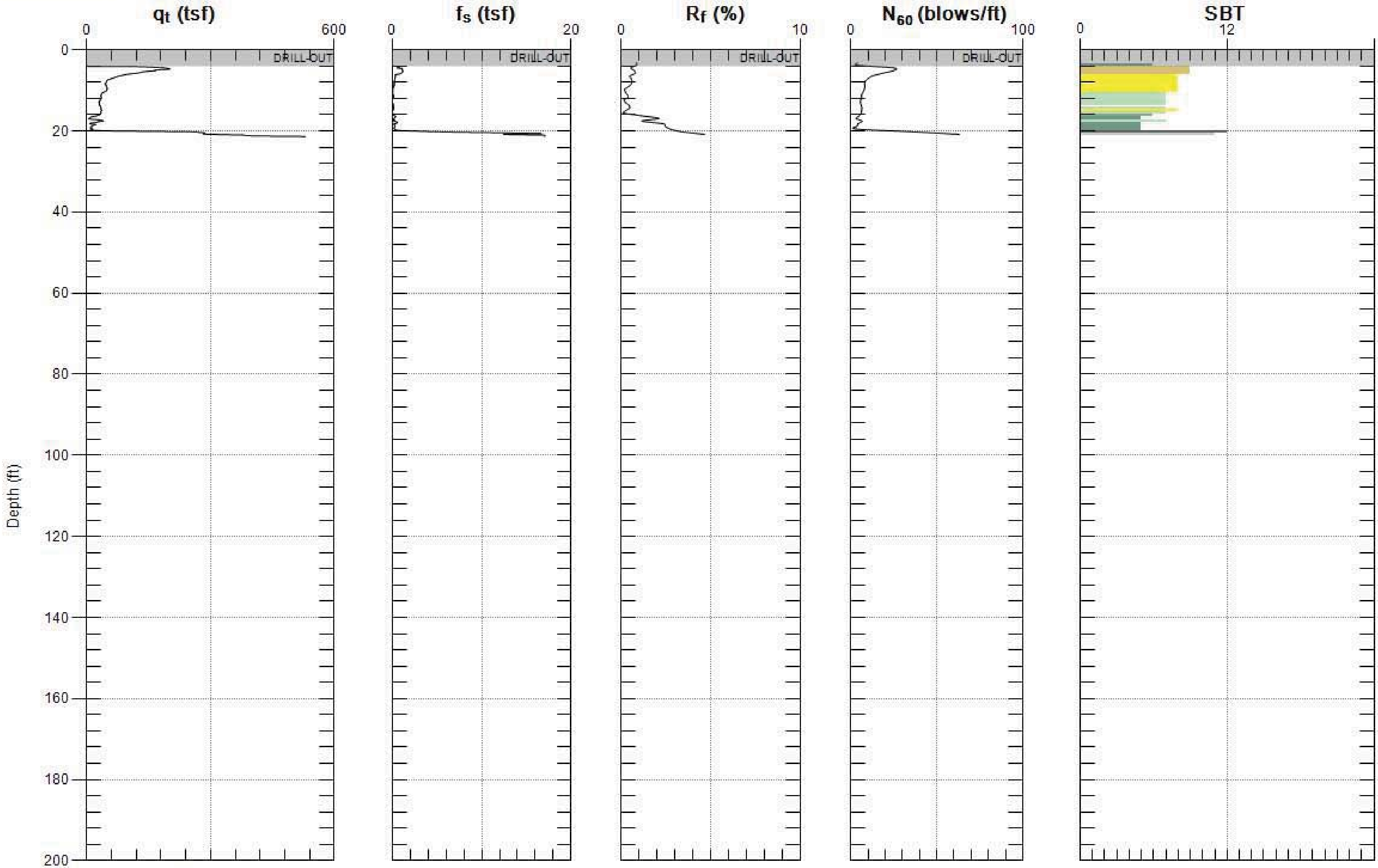
Max. Depth: 190.617 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-2 Date: 9/18/2008 04:02



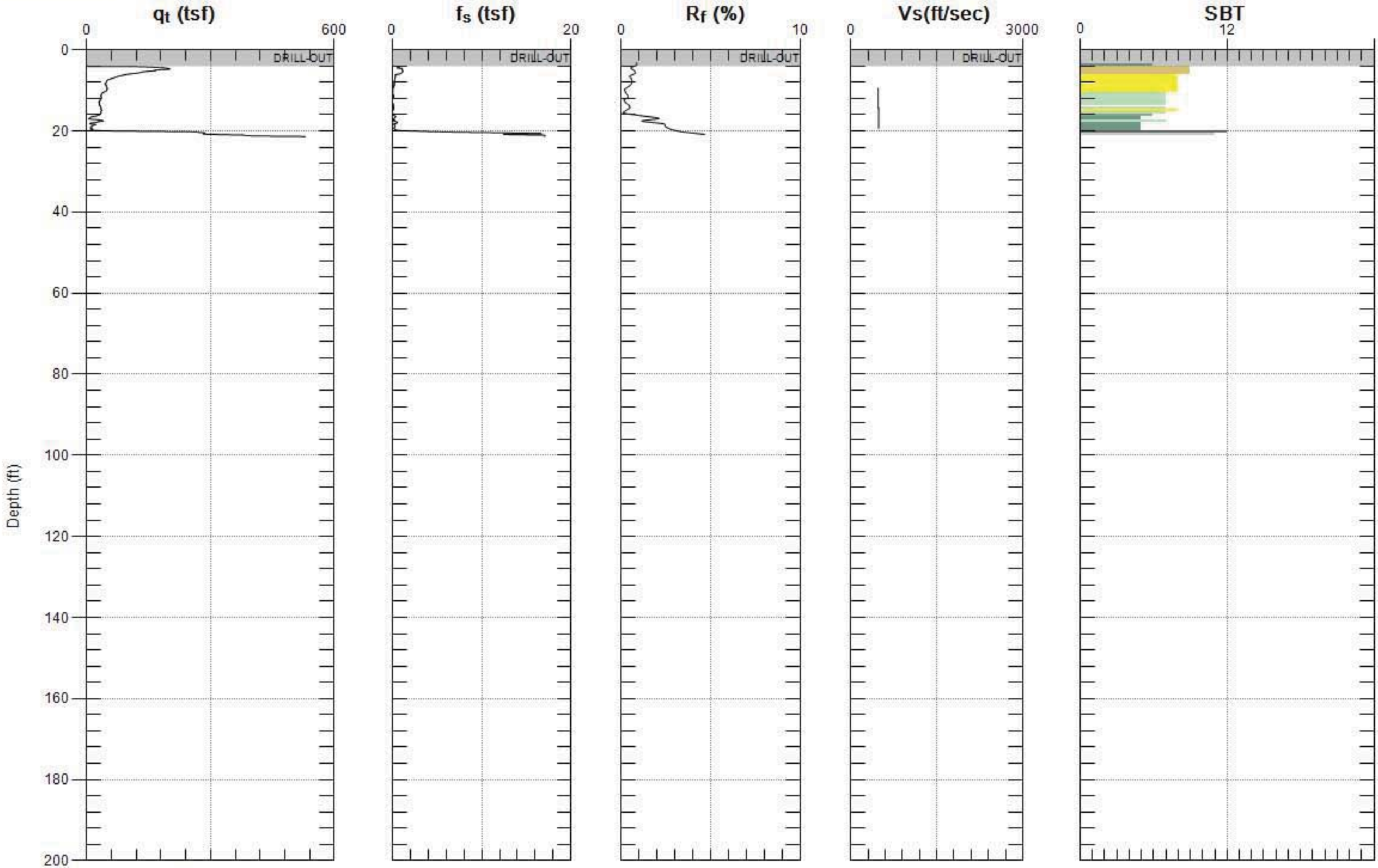
Max. Depth: 21.325 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TERRA ENGINEERS**

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-2 Date: 9/18/2008 04:02



Max. Depth: 21.325 (ft)
Avg. Interval: 0.656 (ft)

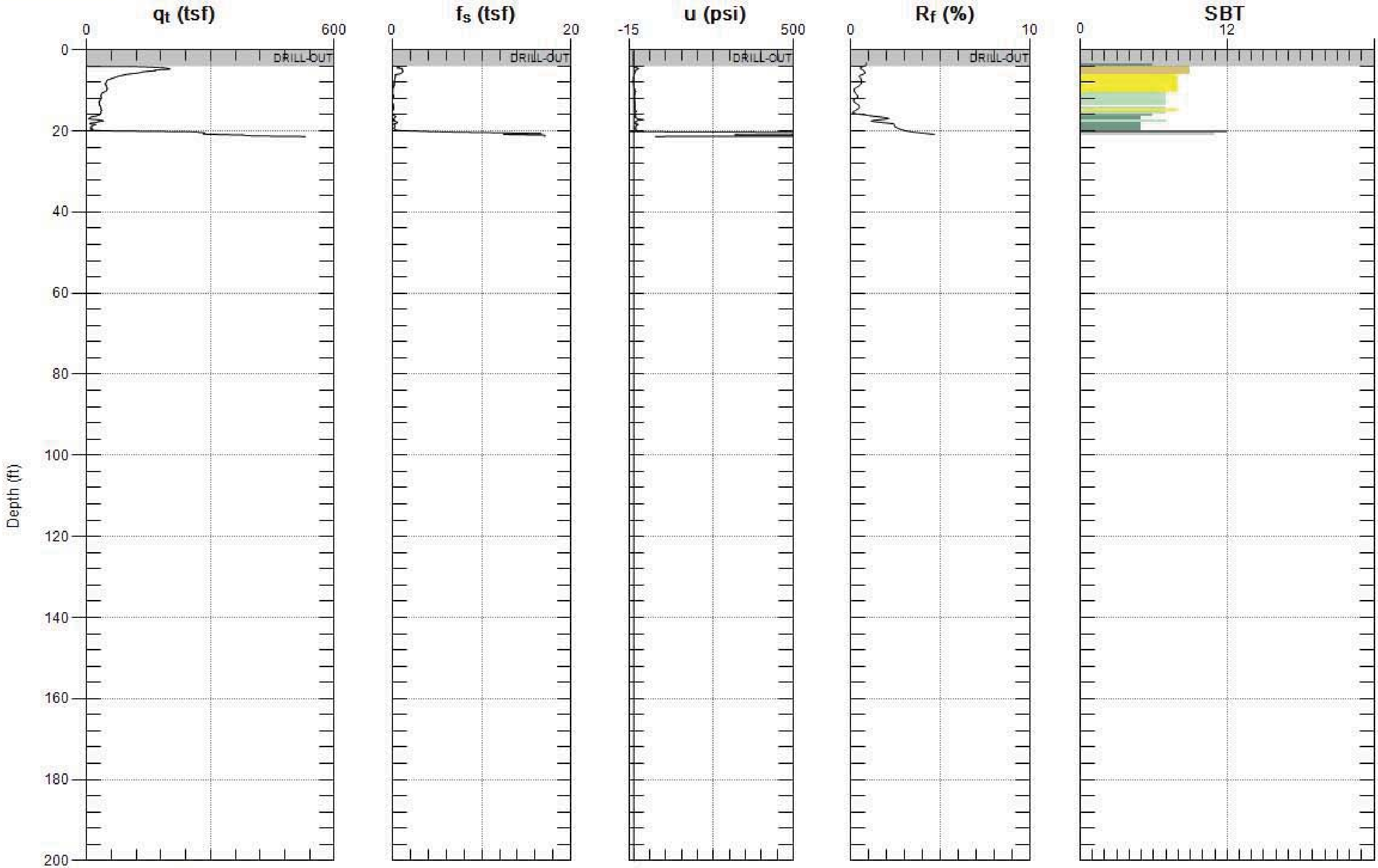
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-2 Date: 9/18/2008 04:02



Max. Depth: 21.325 (ft)
Avg. Interval: 0.656 (ft)

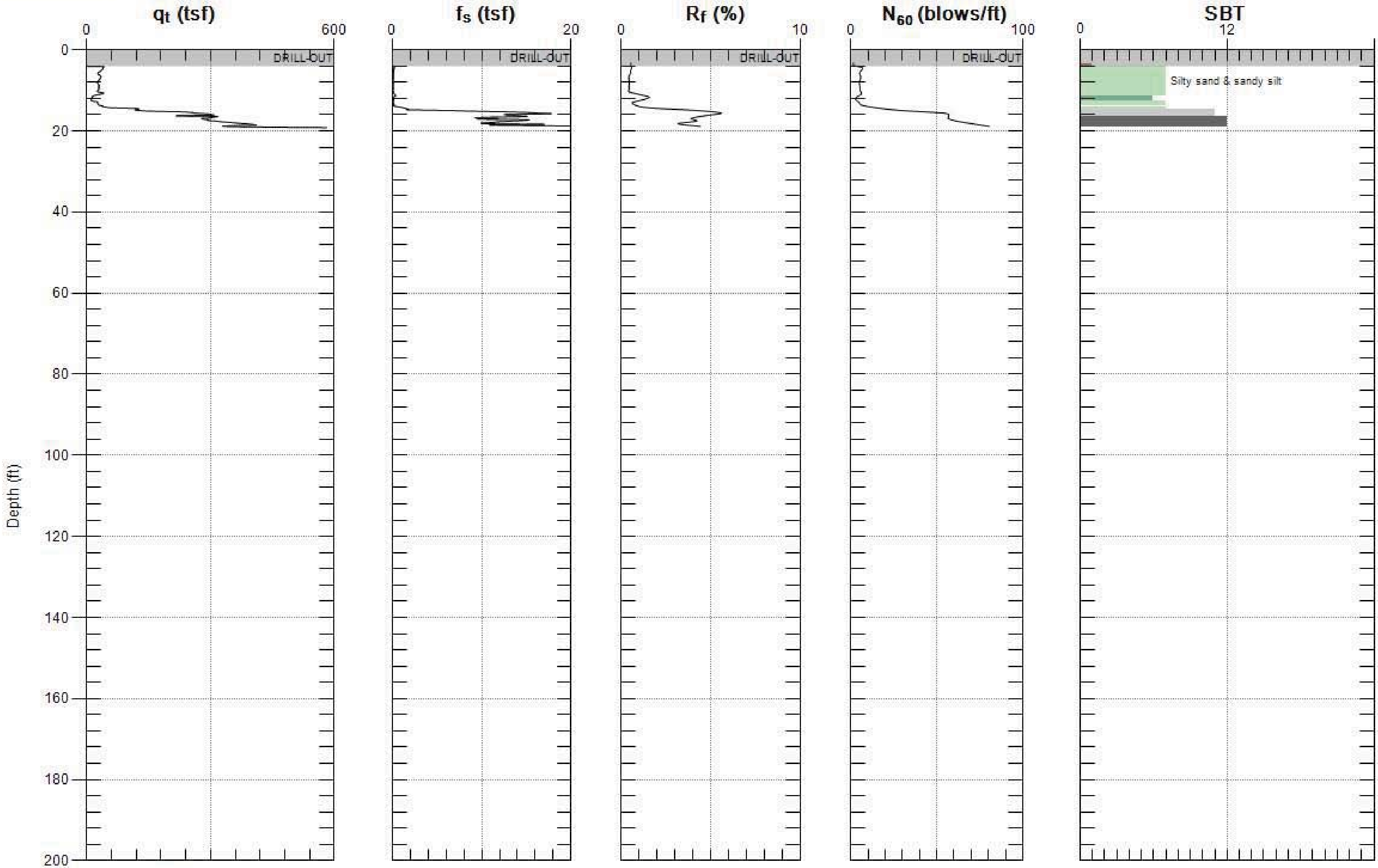
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-3 Date: 9/17/2008 04:05



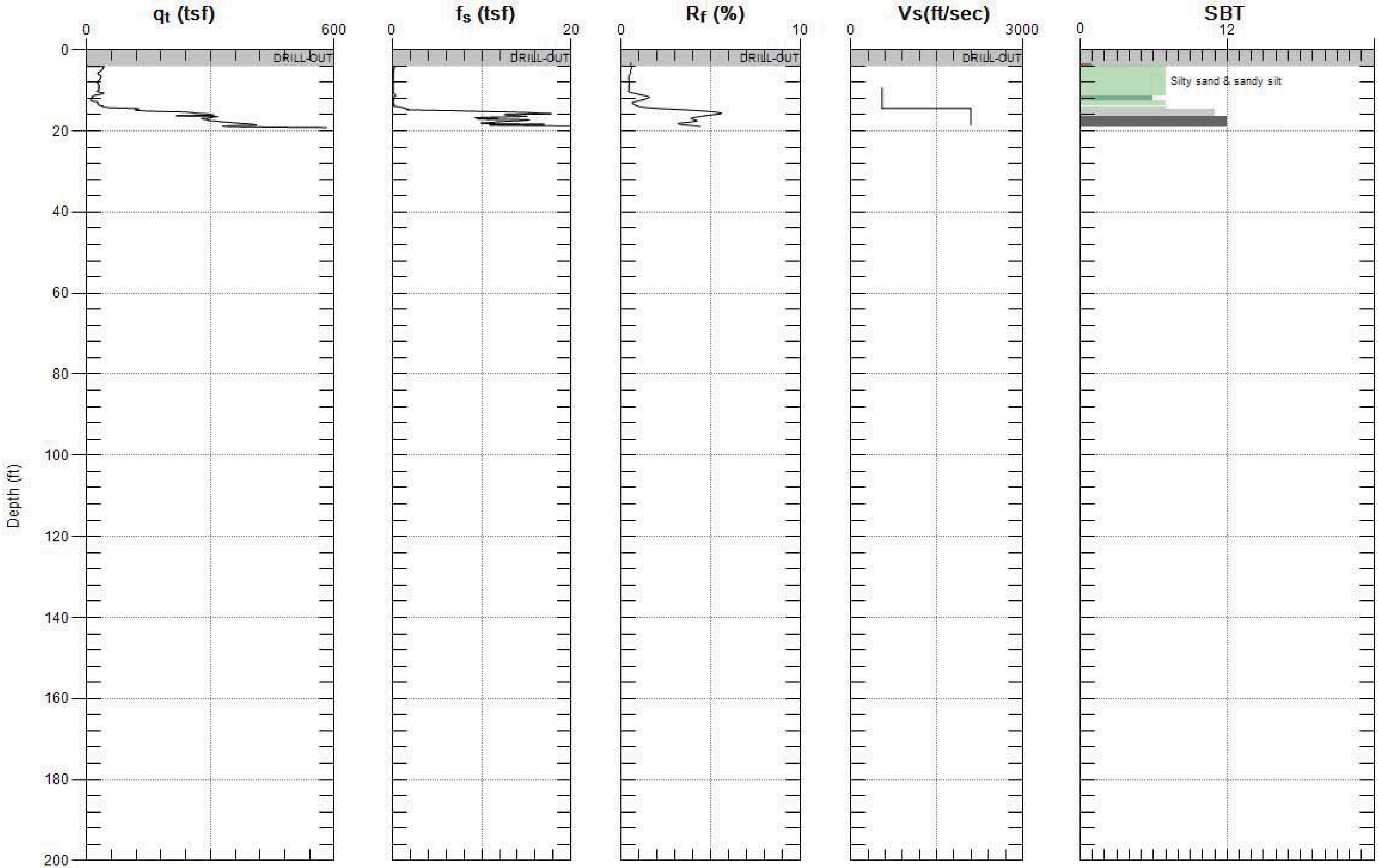
Max. Depth: 19.193 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-3 Date: 9/17/2008 04:05



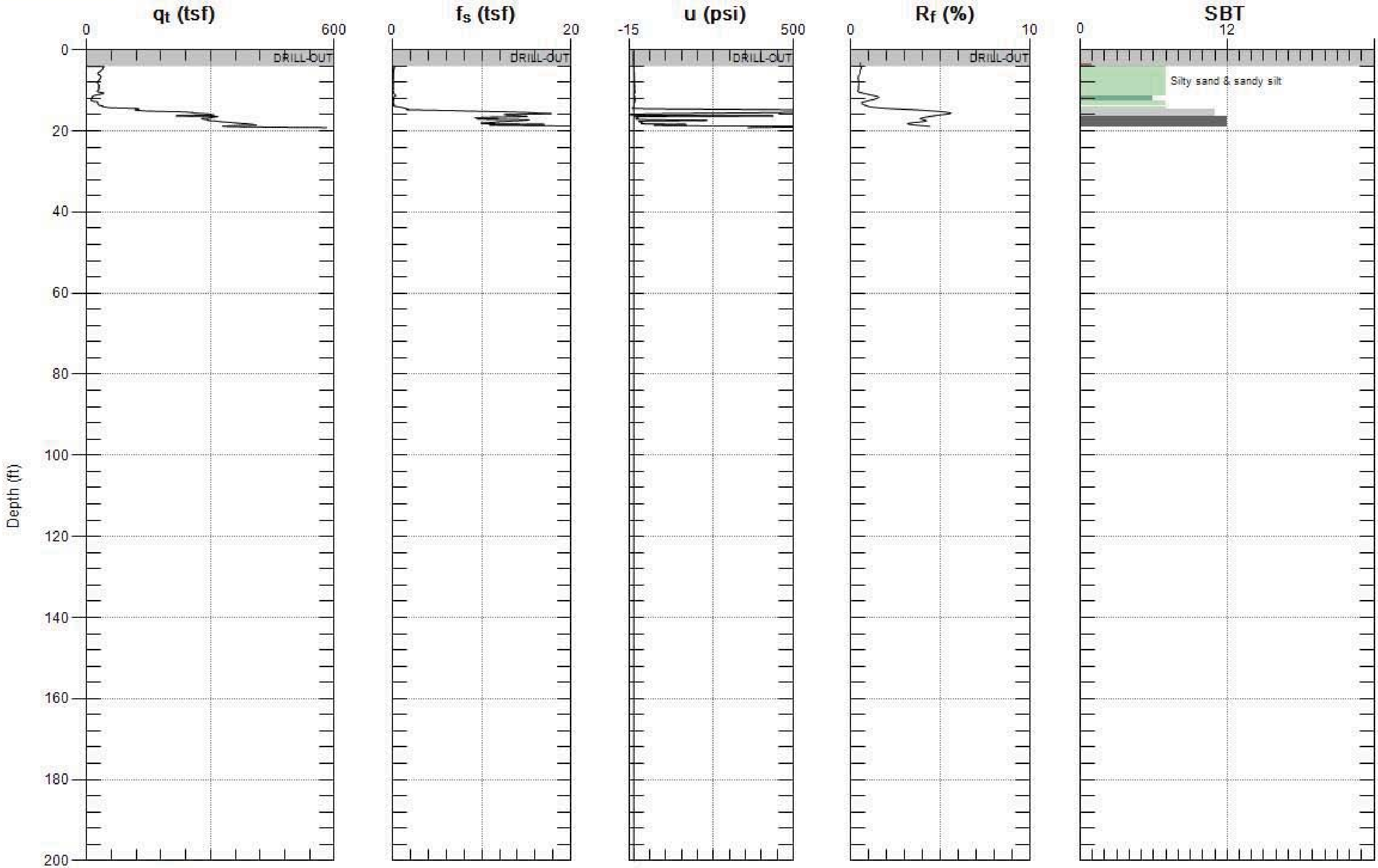
Max. Depth: 19.193 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-3 Date: 9/17/2008 04:05



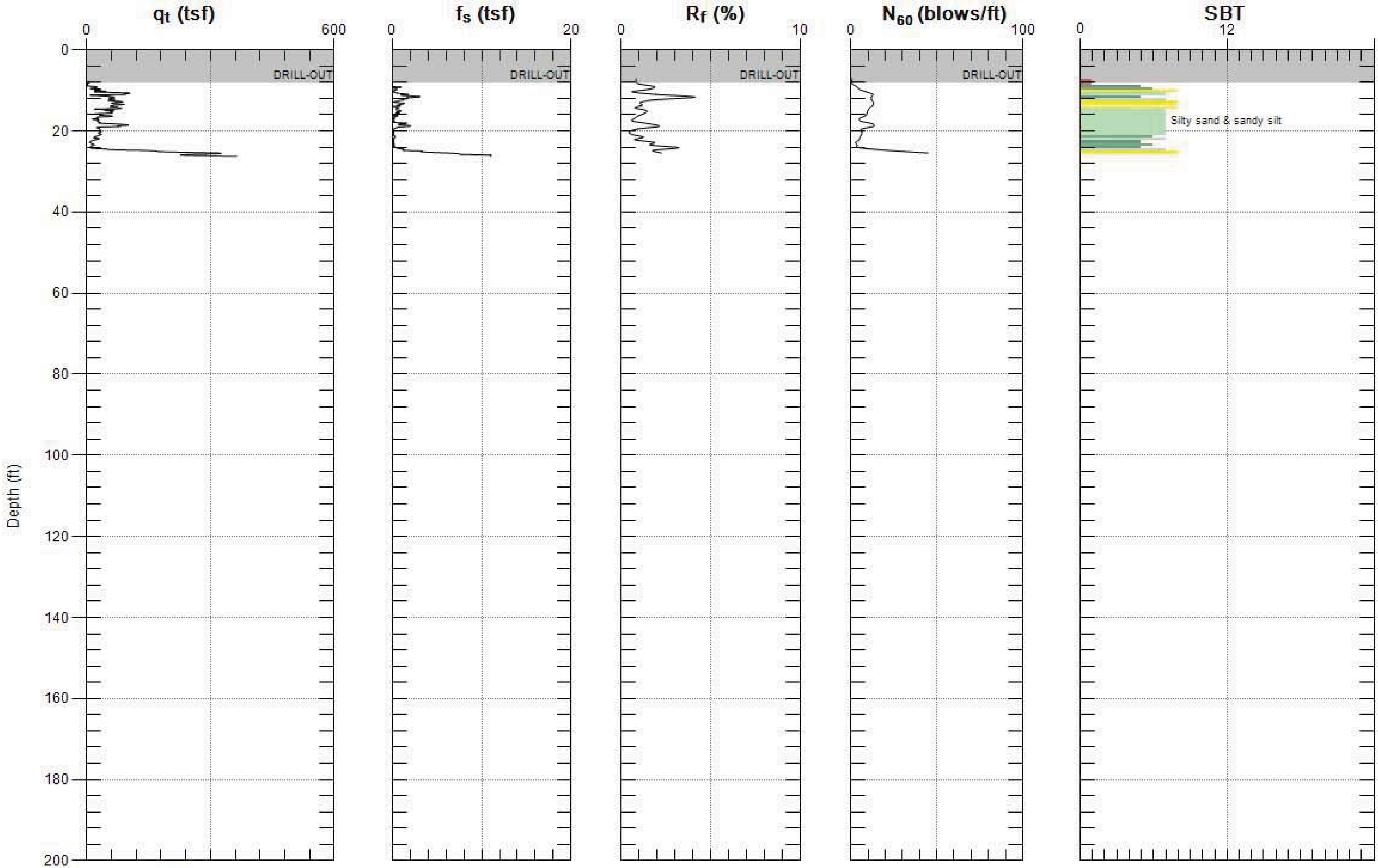
Max. Depth: 19.193 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-4 Date: 9/19/2008 08:58



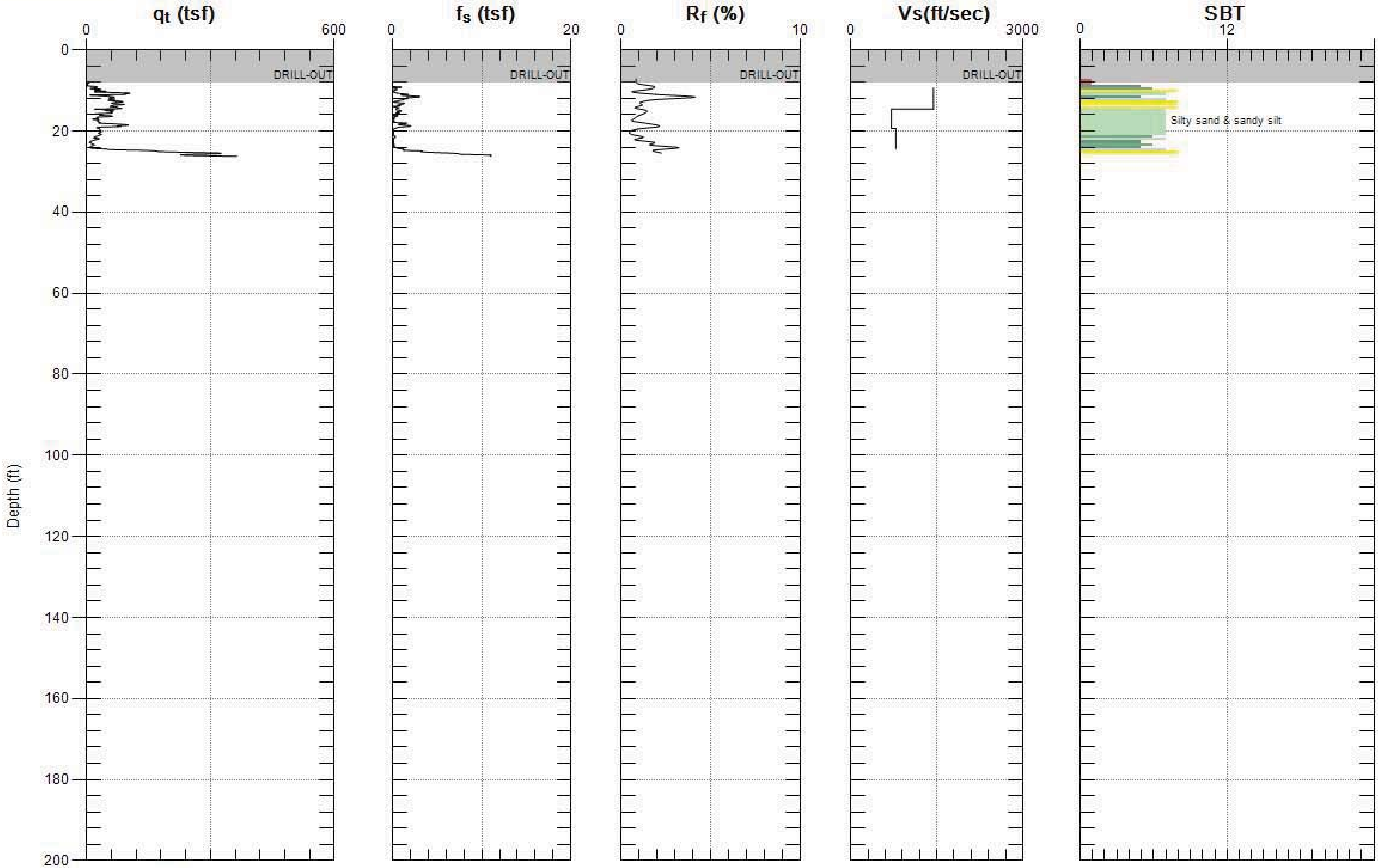
Max. Depth: 26.247 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TERRA ENGINEERS**

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-4 Date: 9/19/2008 08:58



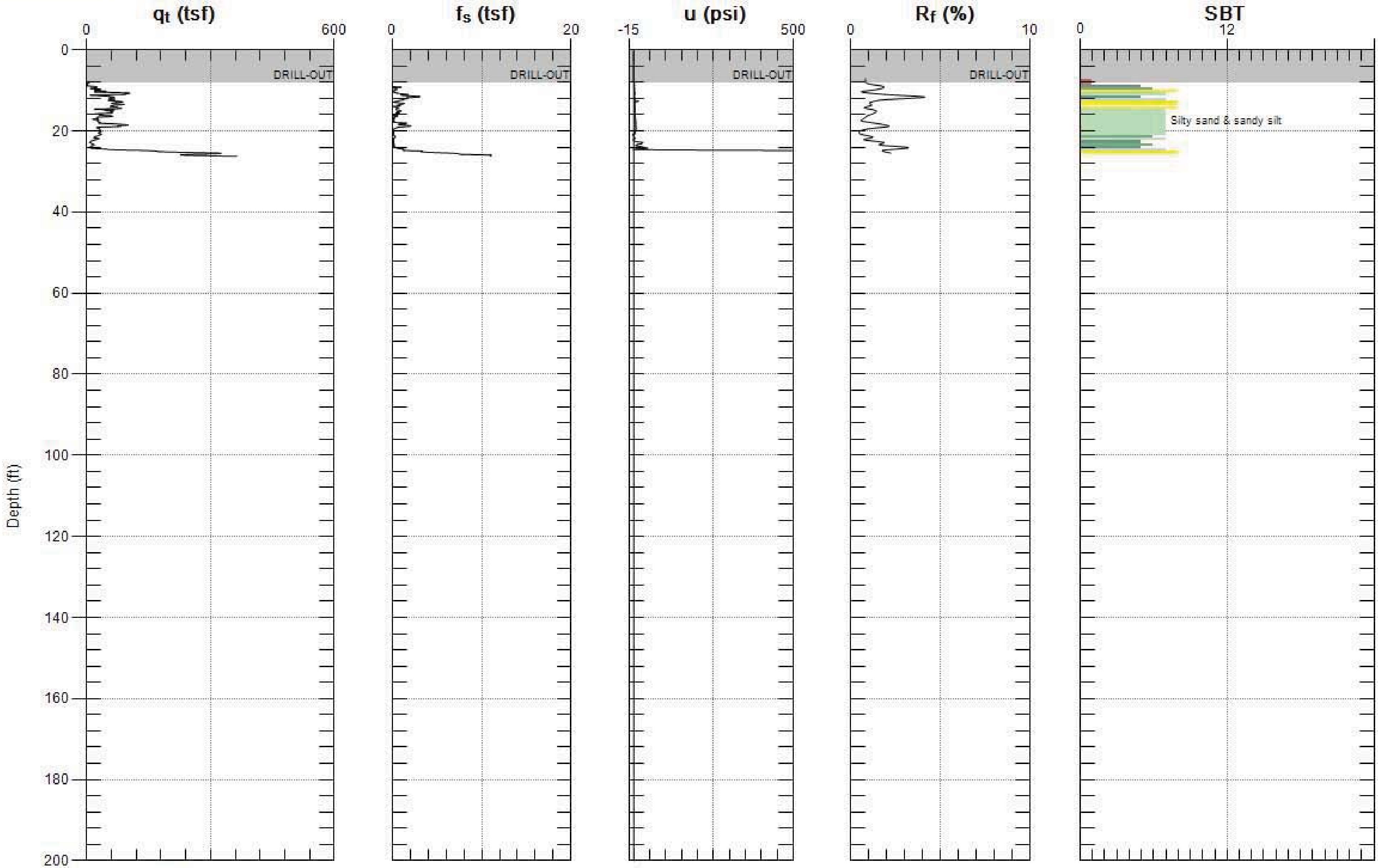
Max. Depth: 26.247 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-4 Date: 9/19/2008 08:58



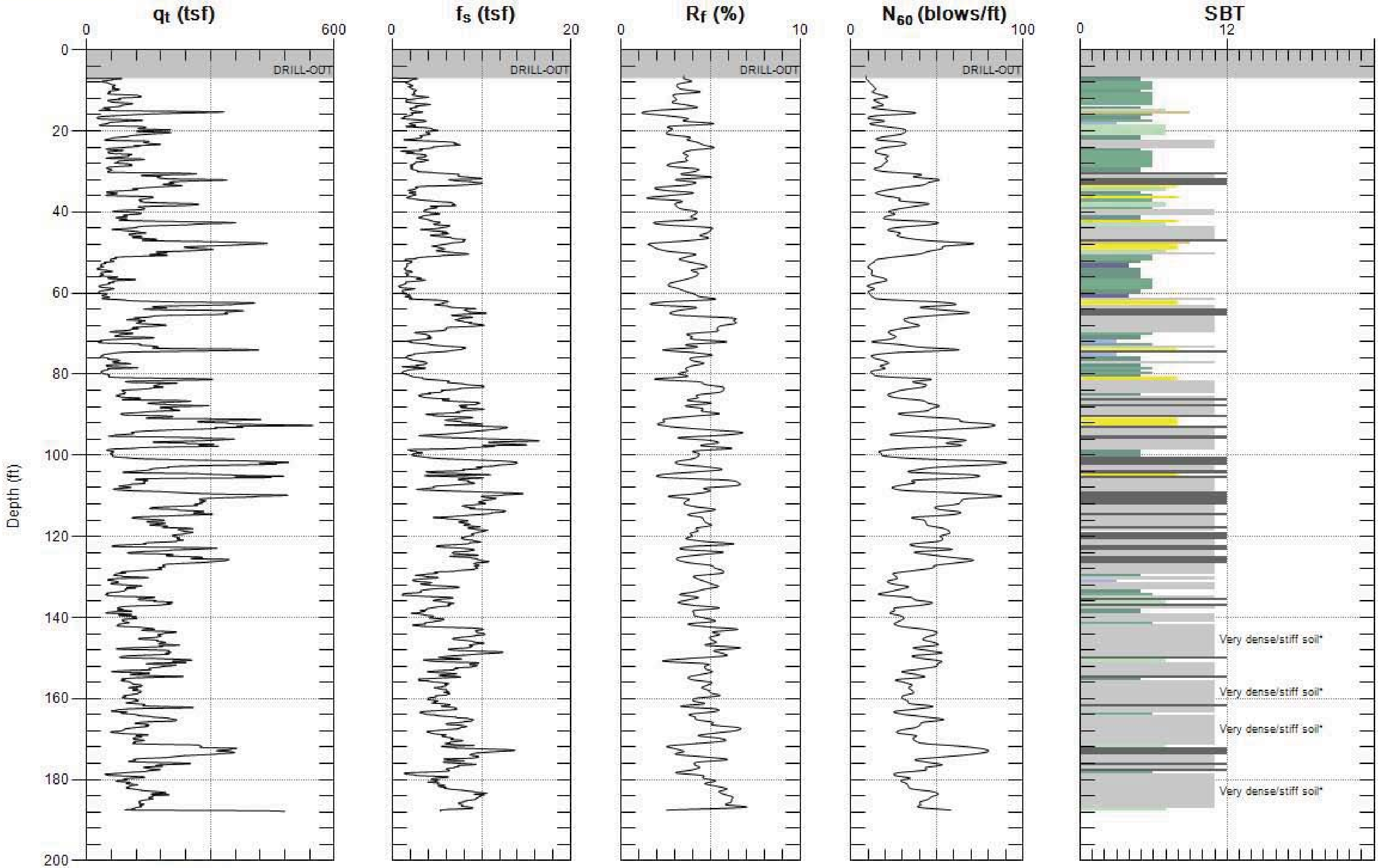
Max. Depth: 26.247 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-6 Date: 9/16/2008 08:44



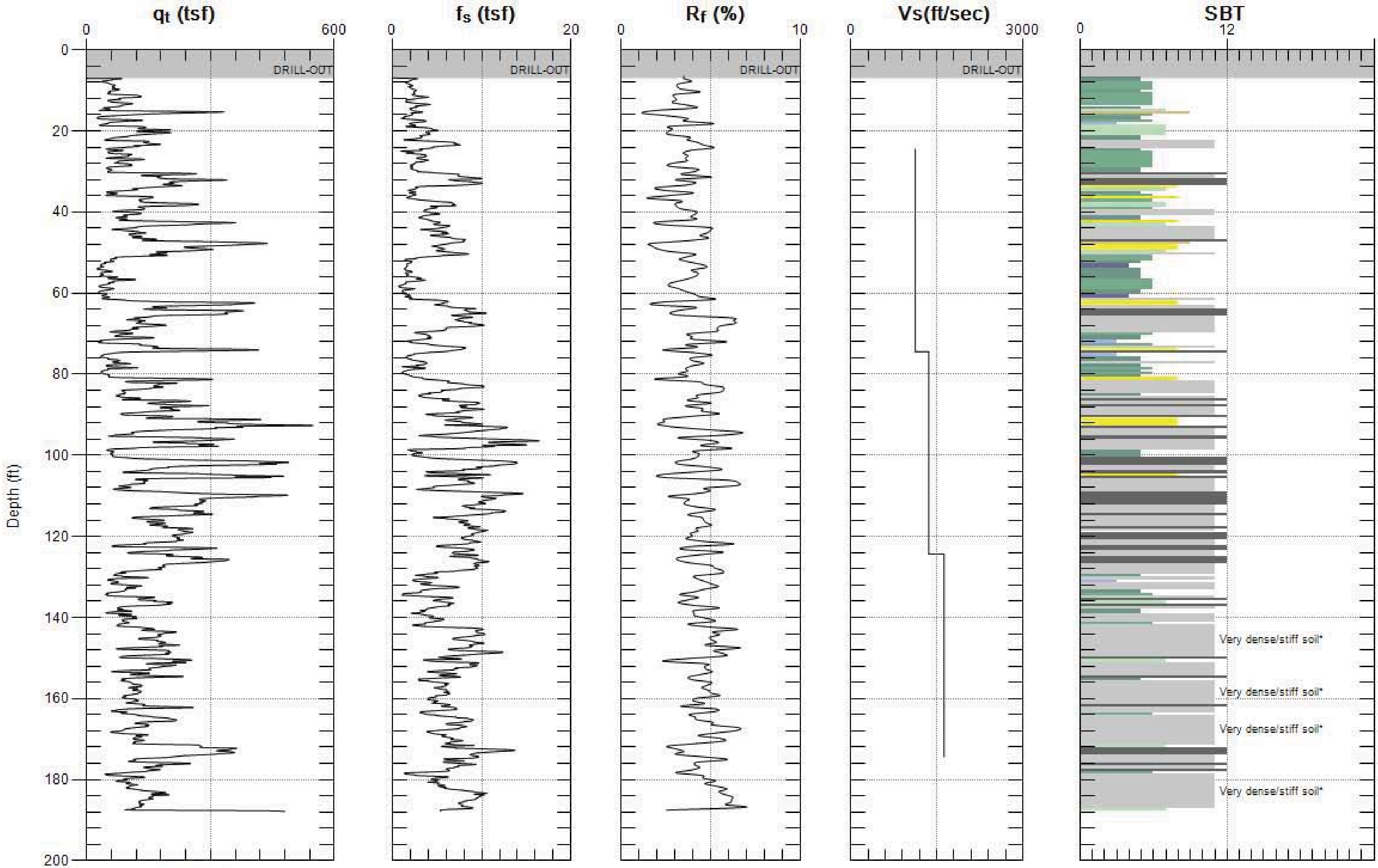
Max. Depth: 187.828 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-6 Date: 9/16/2008 08:44



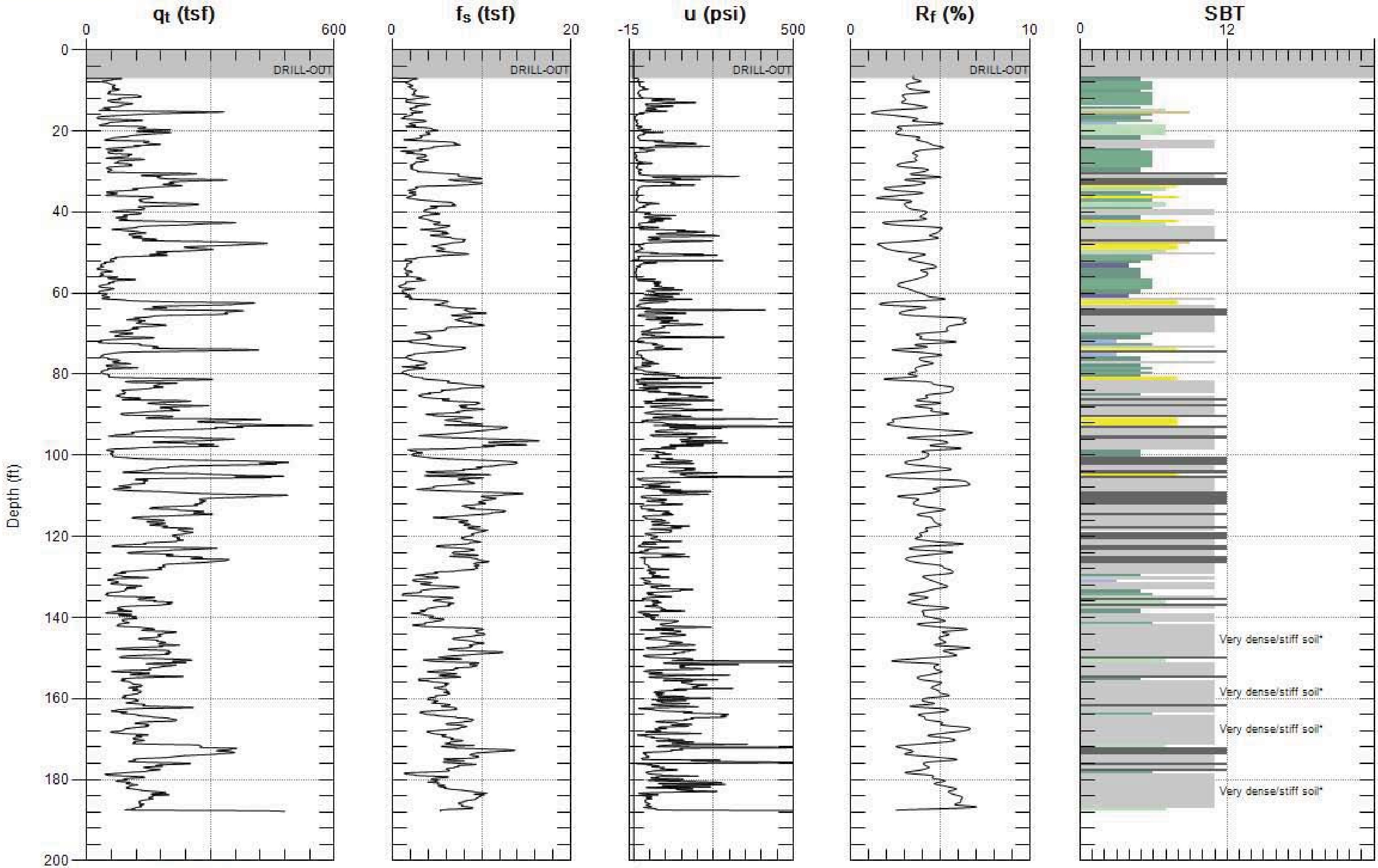
Max. Depth: 187.828 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-6 Date: 9/16/2008 08:44



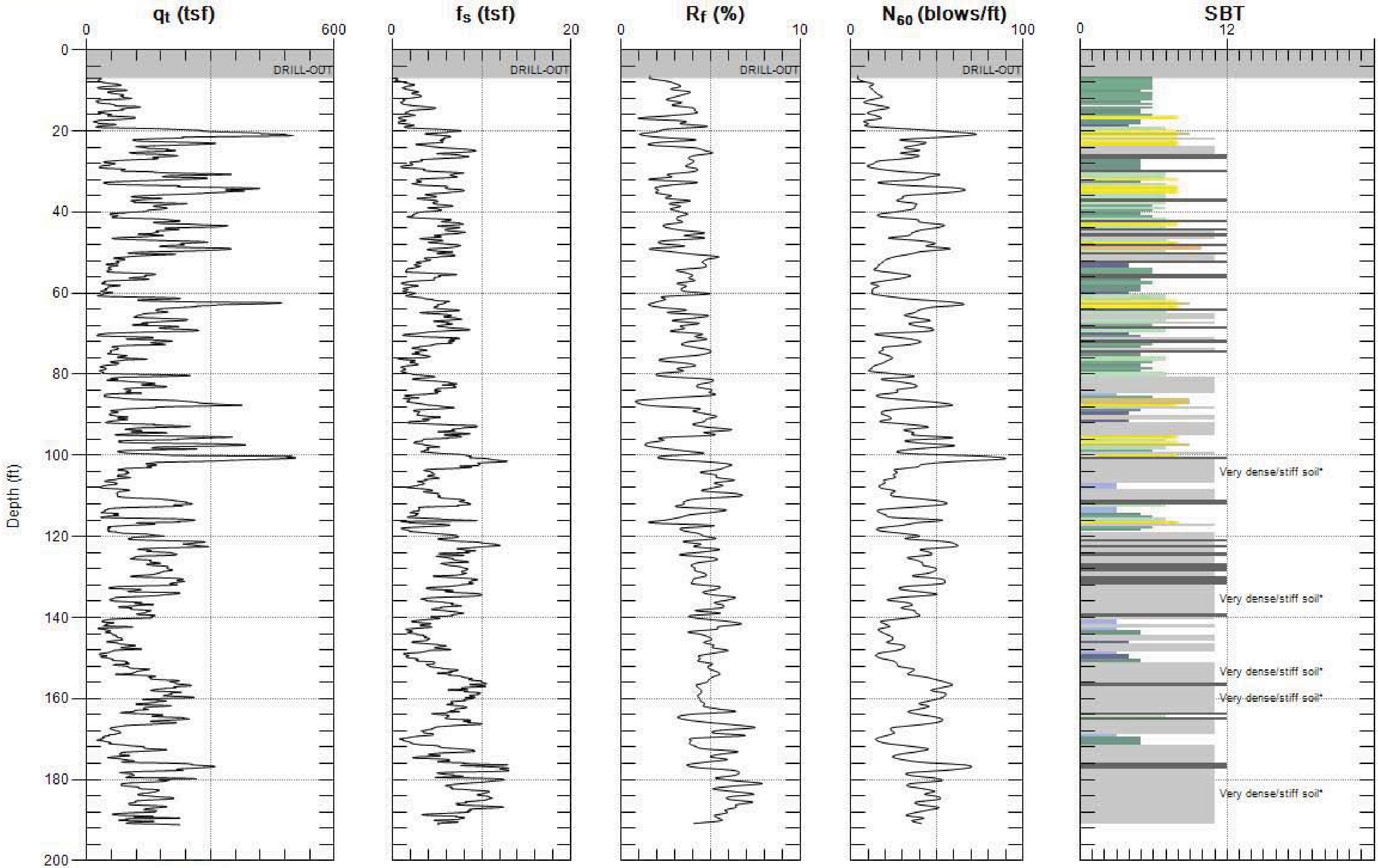
Max. Depth: 187.828 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-7 Date: 9/15/2008 09:46



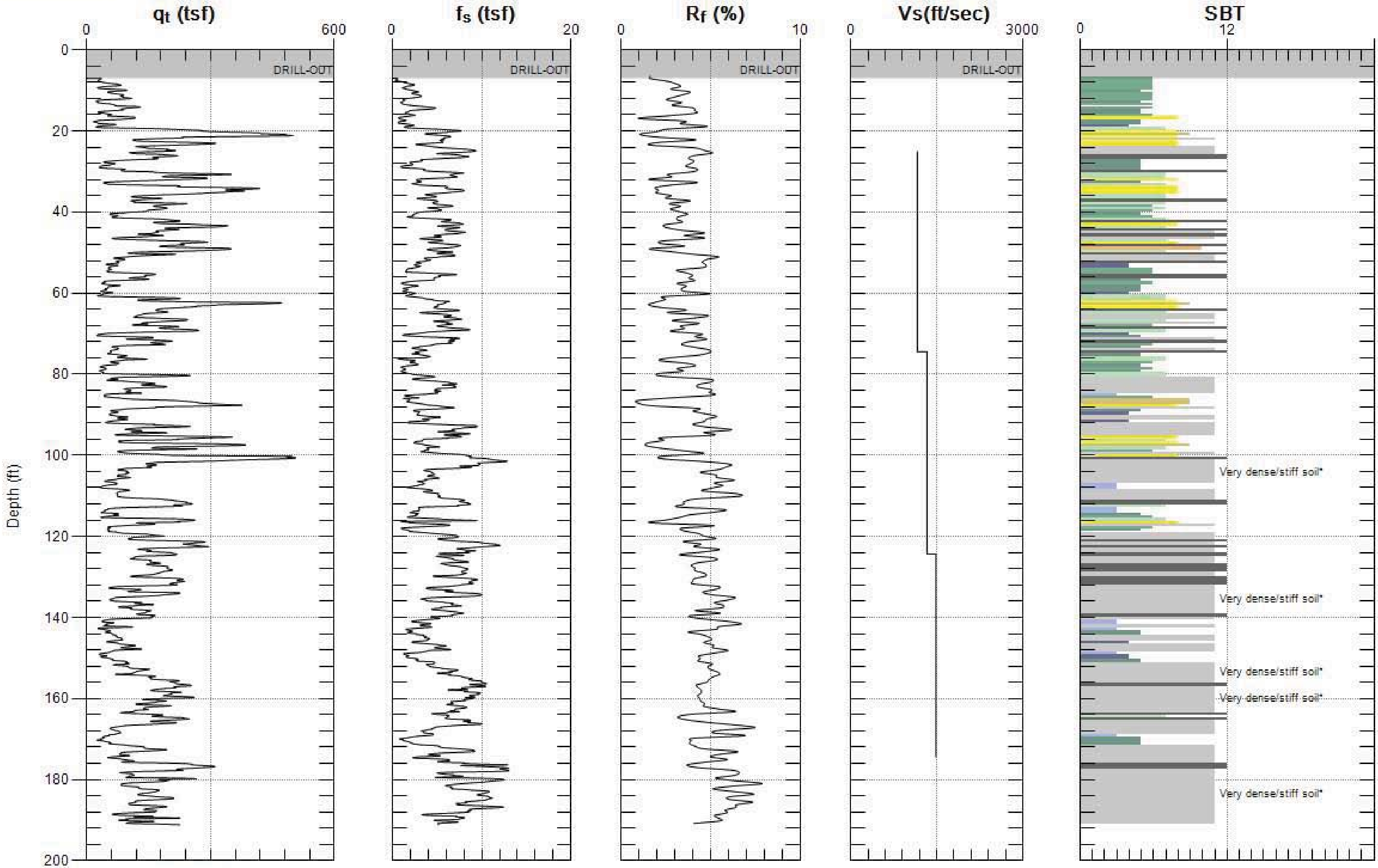
Max. Depth: 191.109 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-7 Date: 9/15/2008 09:46



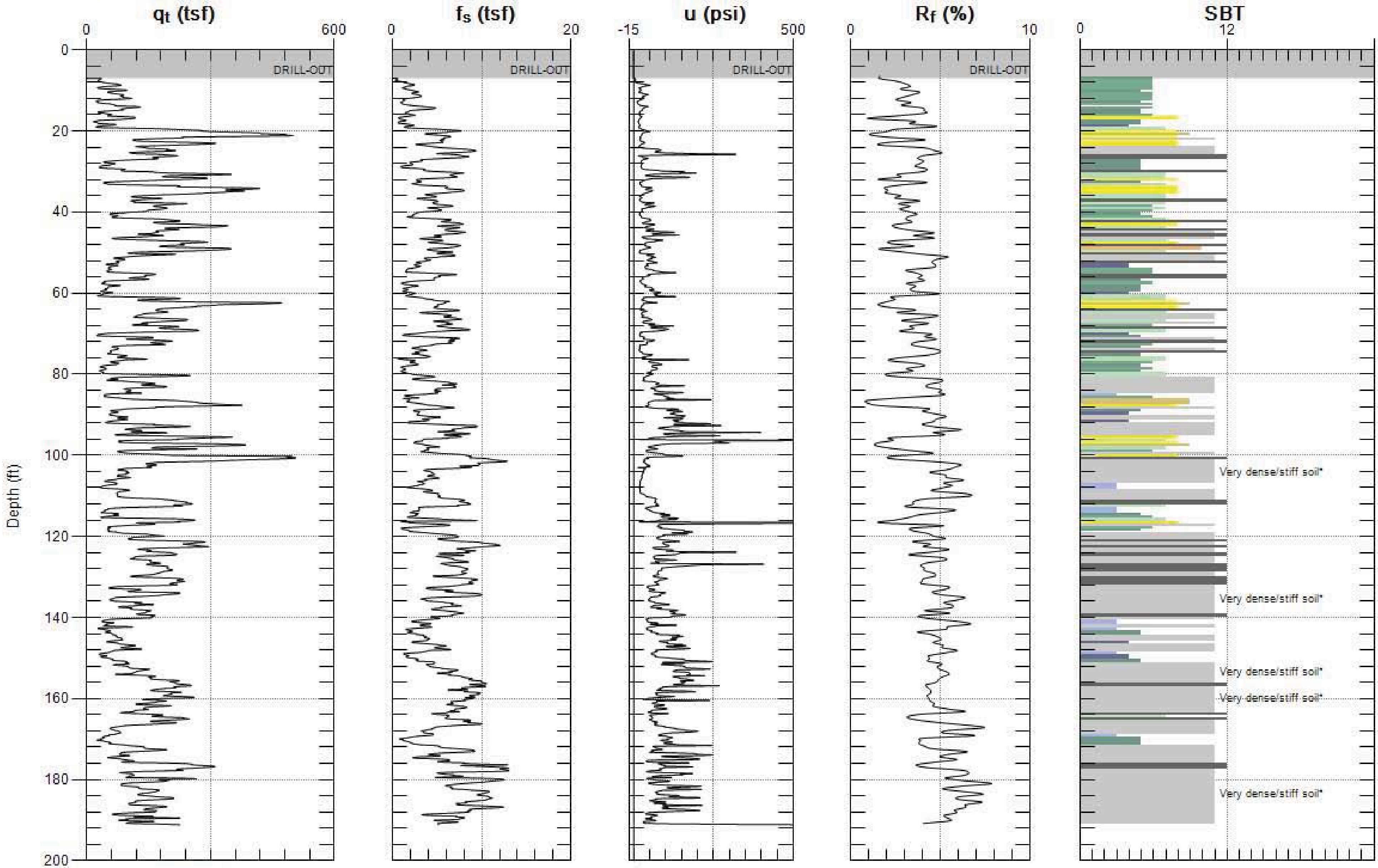
Max. Depth: 191.109 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TERRA ENGINEERS**

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-7 Date: 9/15/2008 09:46



Max. Depth: 191.109 (ft)
Avg. Interval: 0.656 (ft)

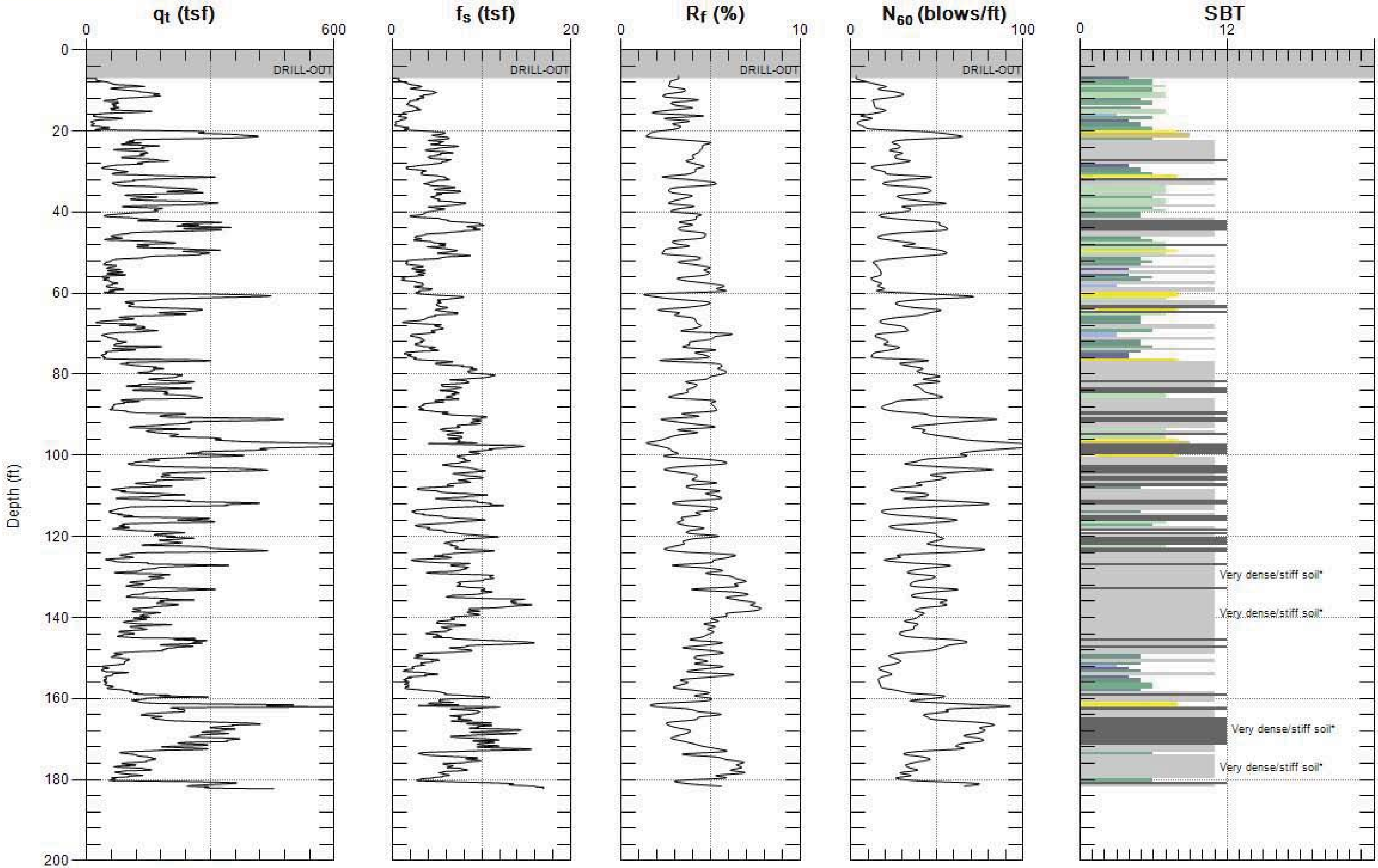
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR **Engineer: R.HARLAN**

Sounding: MC-8 **Date: 9/16/2008 02:01**



Max. Depth: 182.251 (ft)
Avg. Interval: 0.656 (ft)

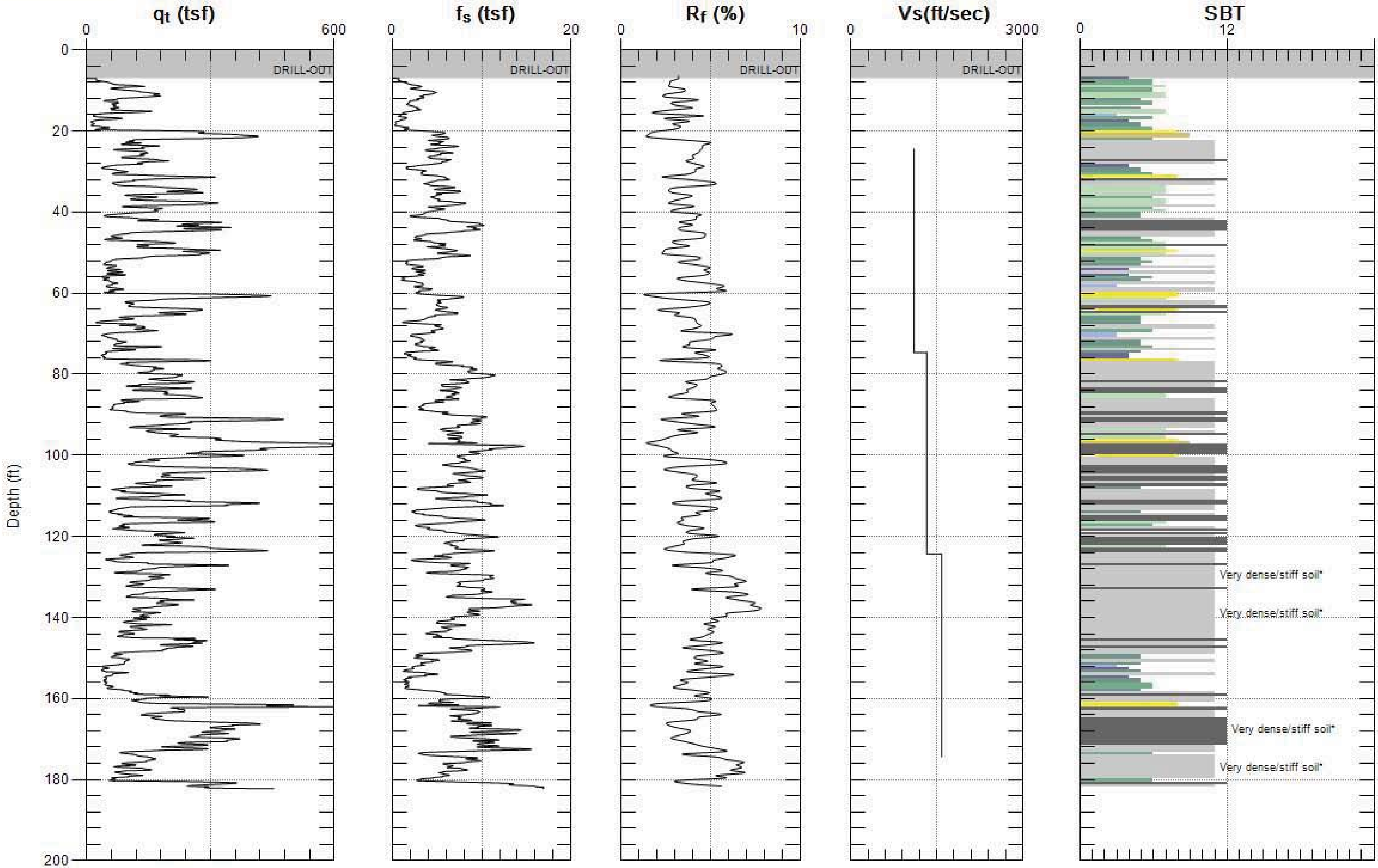
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-8 Date: 9/16/2008 02:01

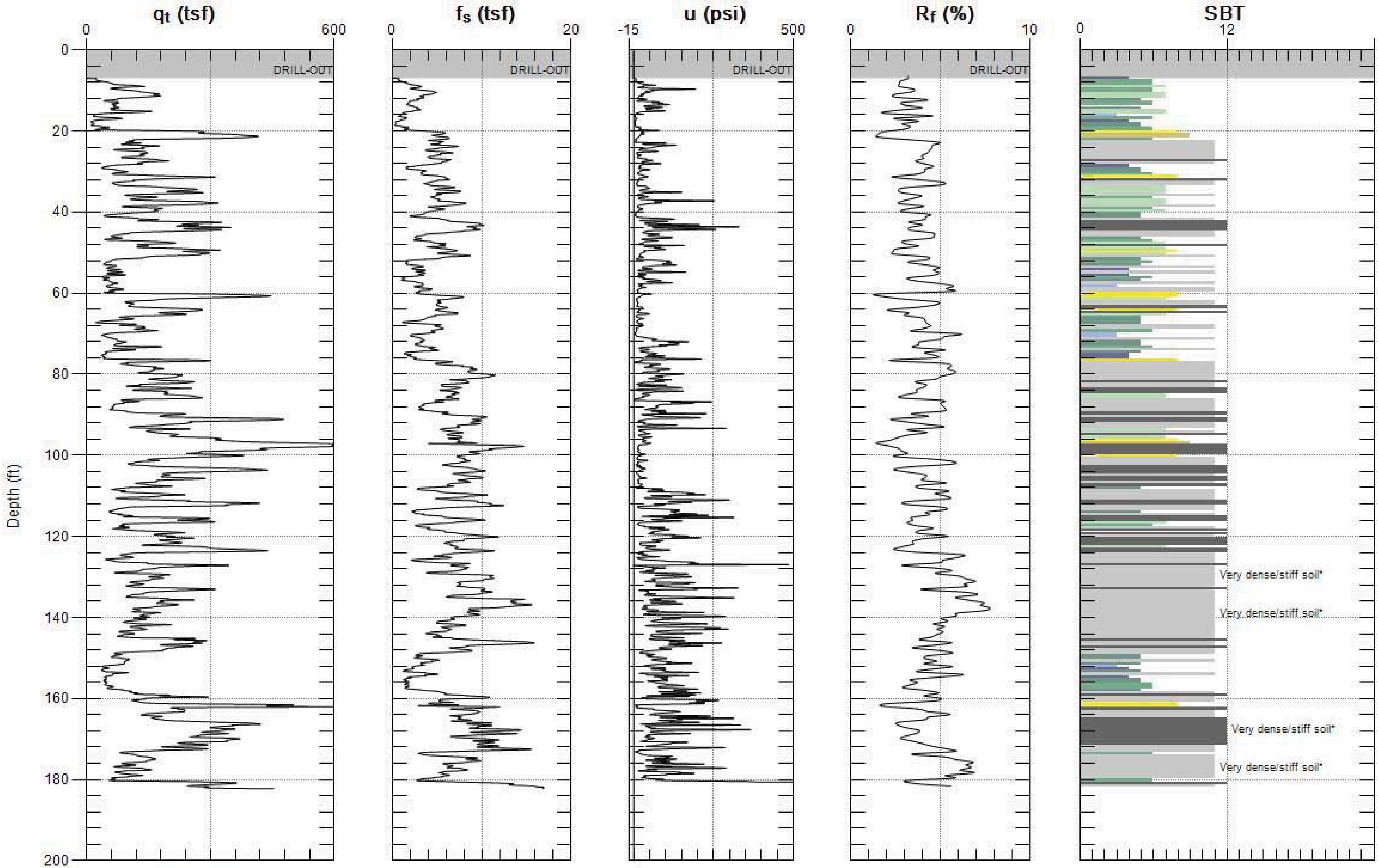


Max. Depth: 182.251 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TERRA ENGINEERS**

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN
Sounding: MC-8 Date: 9/16/2008 02:01



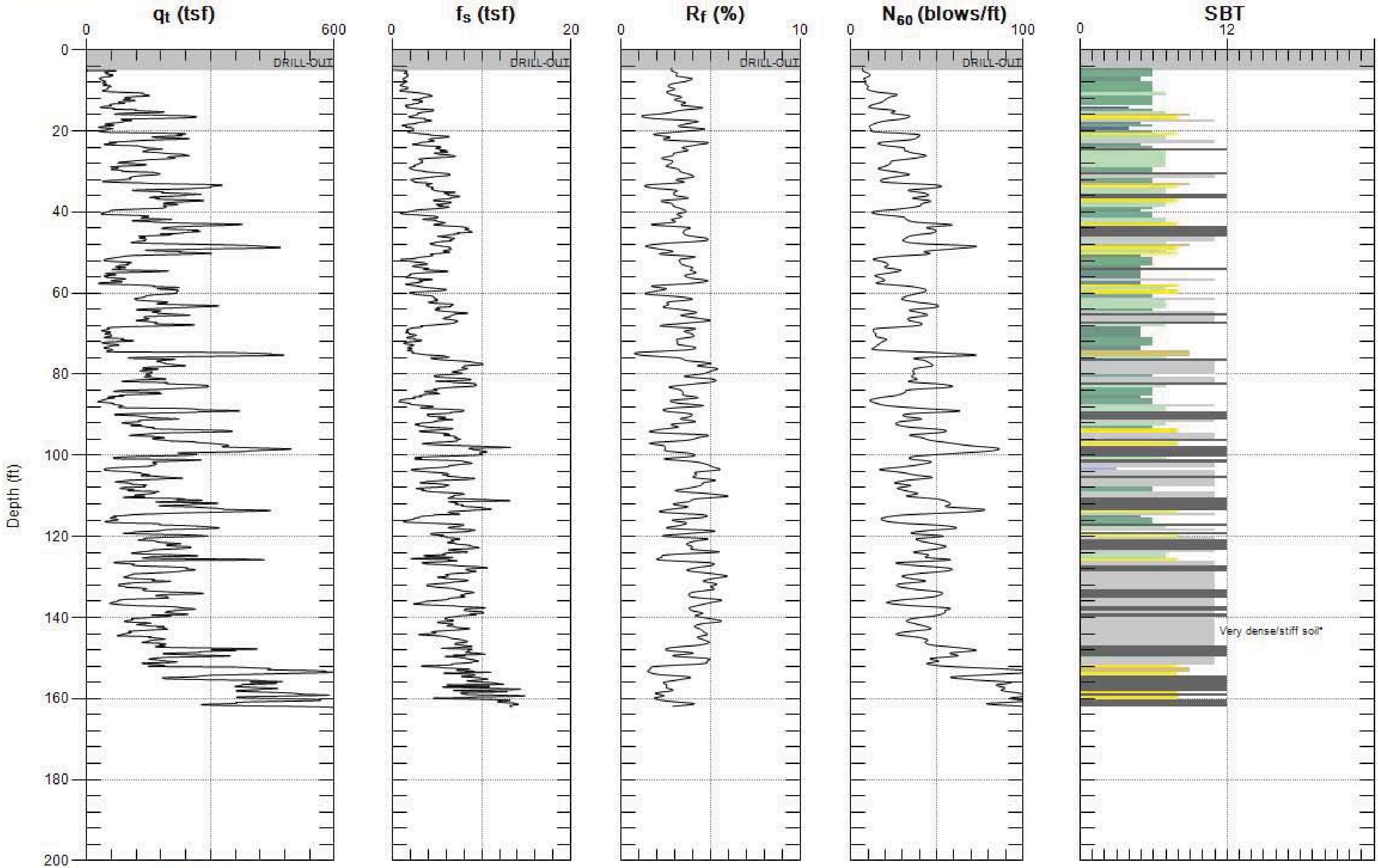
Max. Depth: 182.251 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-9 Date: 9/17/2008 08:23



Max. Depth: 162.238 (ft)
Avg. Interval: 0.656 (ft)

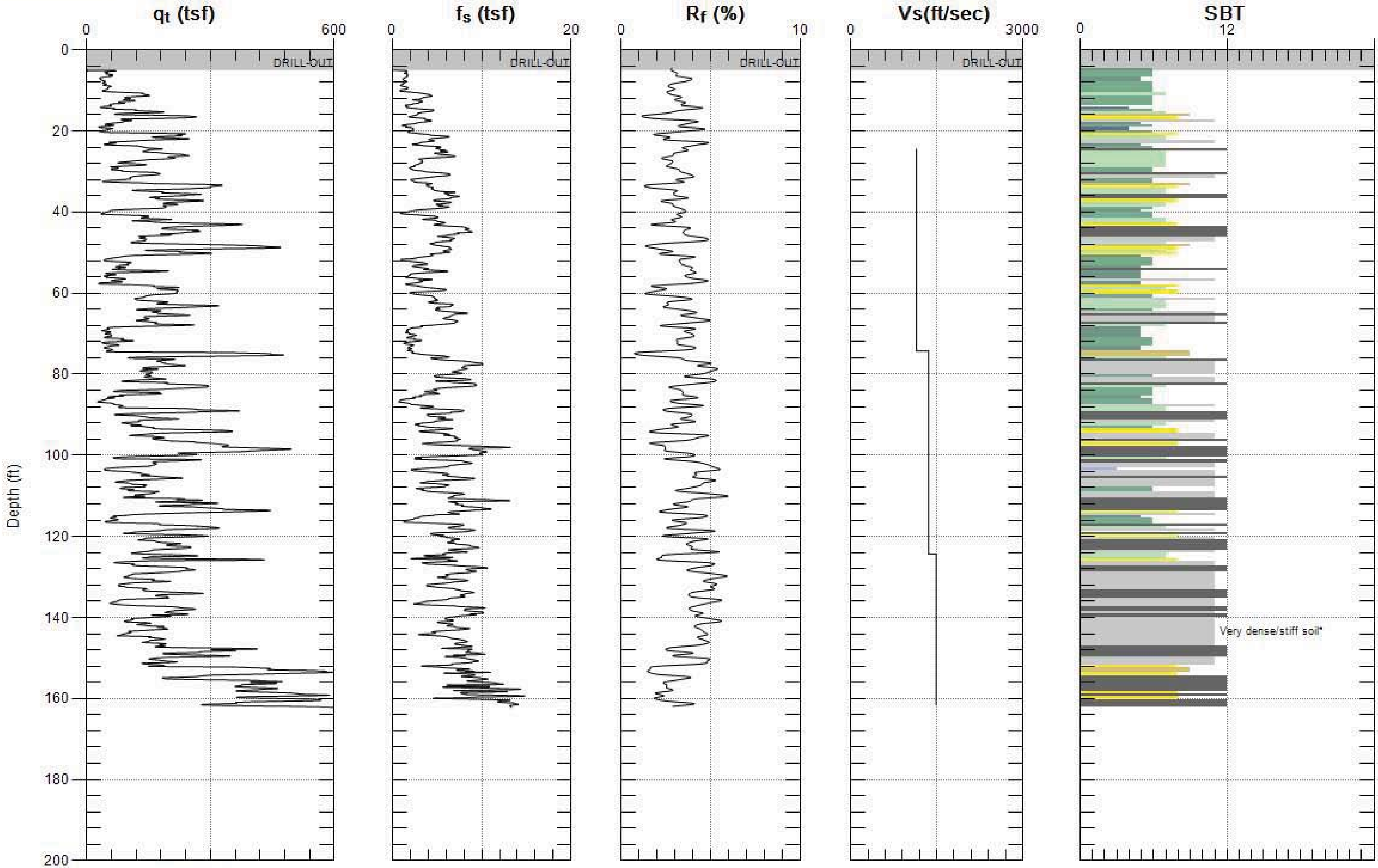
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR **Engineer: R.HARLAN**

Sounding: MC-9 **Date: 9/17/2008 08:23**



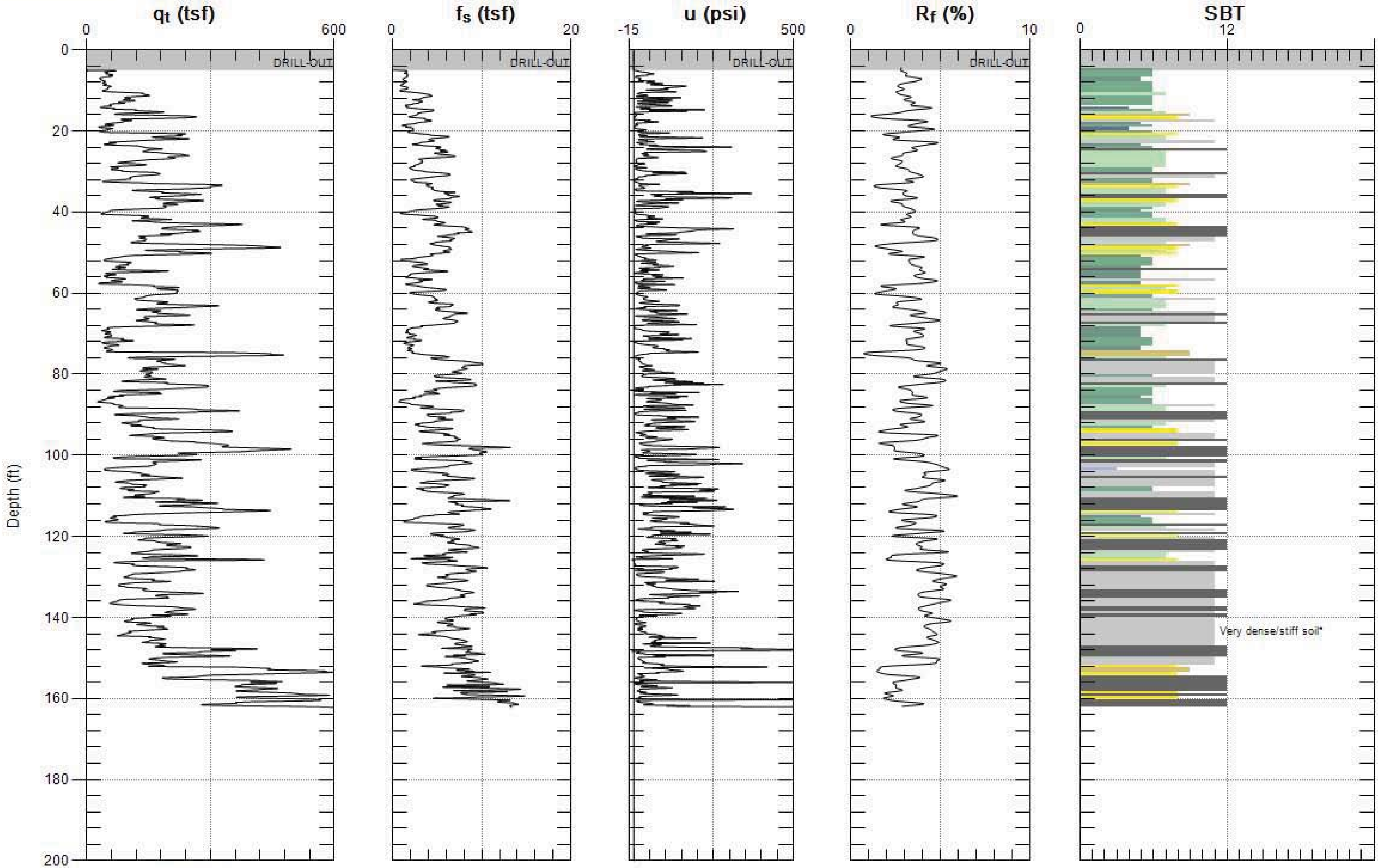
Max. Depth: 162.238 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-9 Date: 9/17/2008 08:23



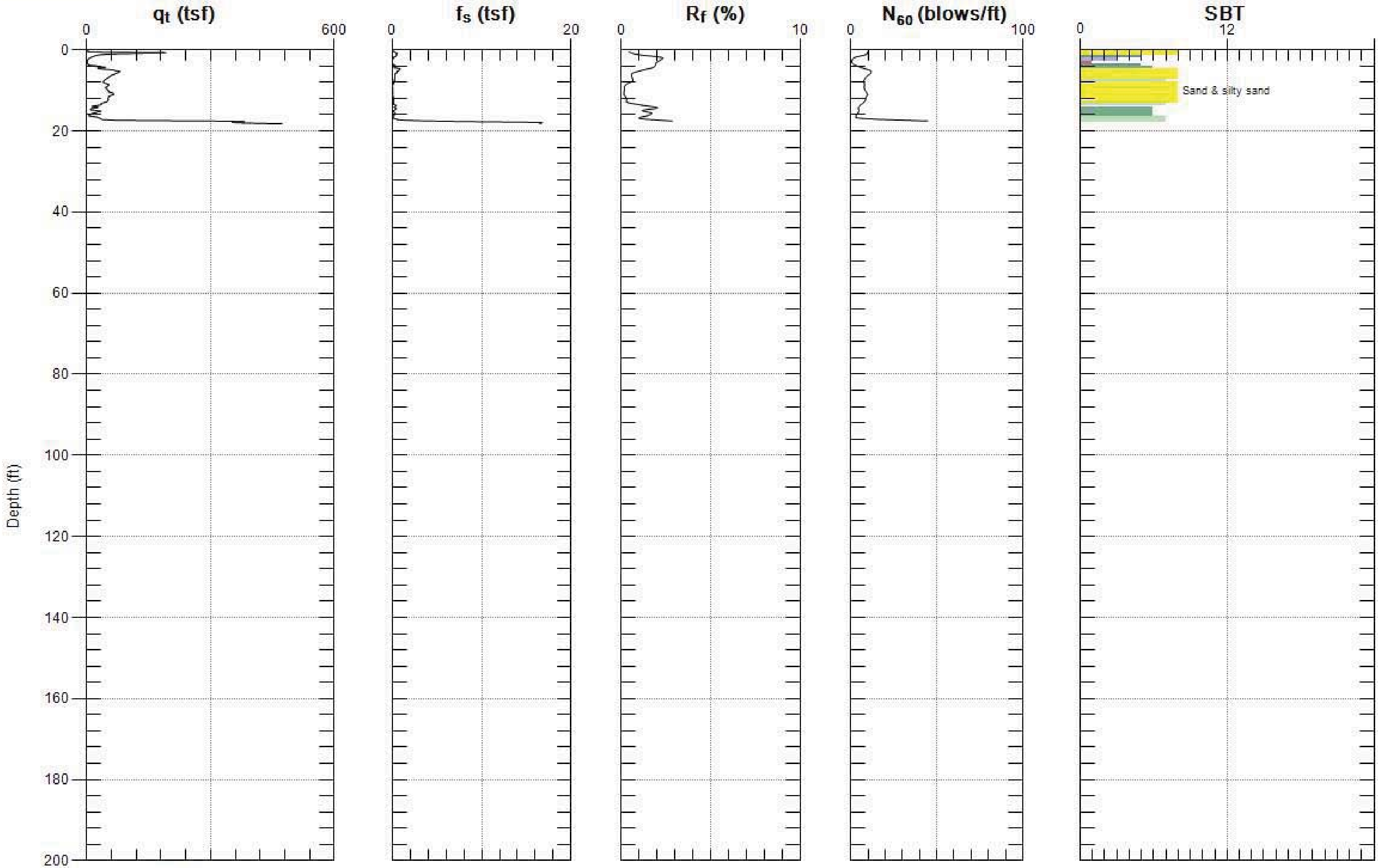
Max. Depth: 162.238 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-10 Date: 9/19/2008 08:14



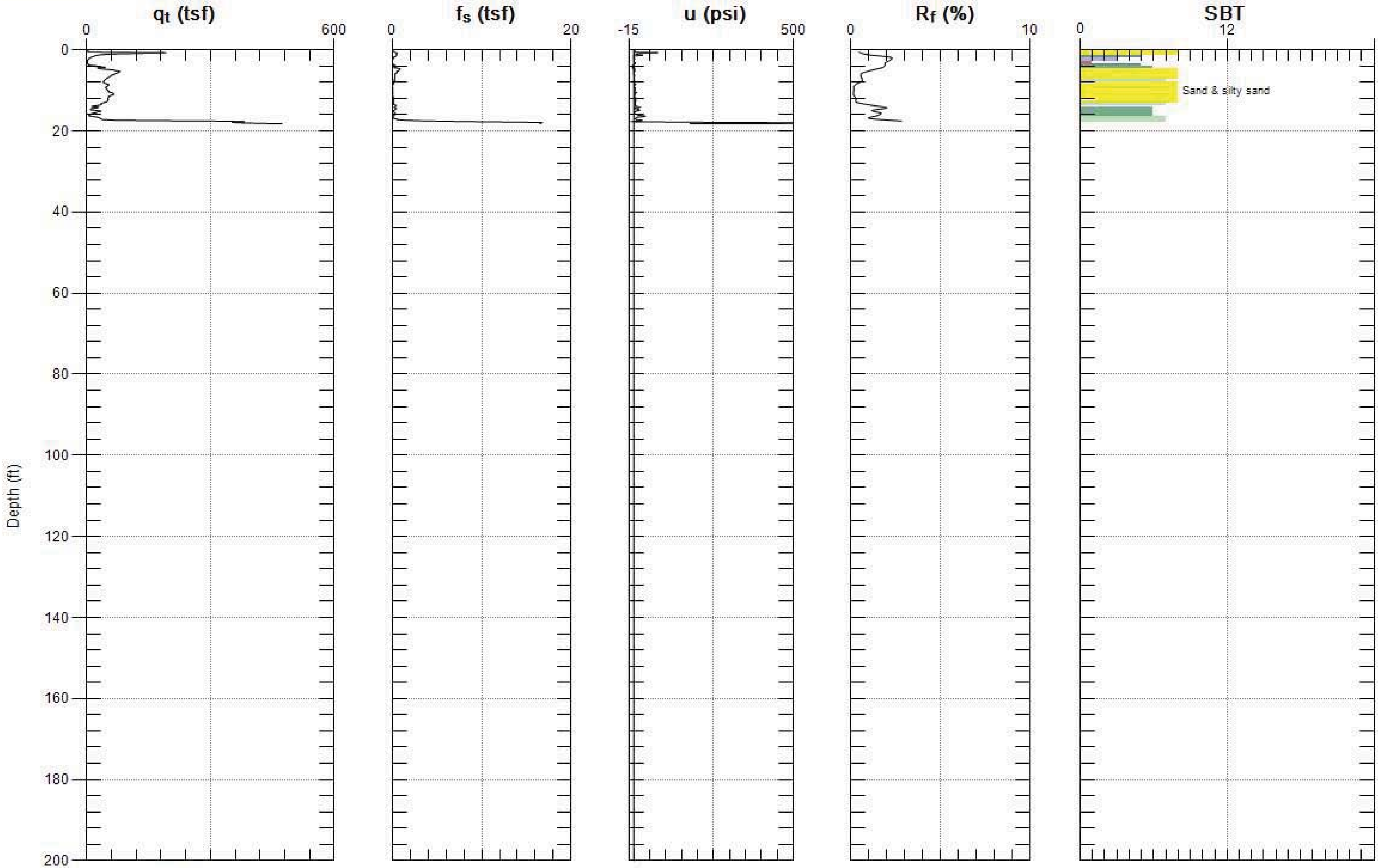
Max. Depth: 18.209 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-10 Date: 9/19/2008 08:14



Max. Depth: 18.209 (ft)
Avg. Interval: 0.656 (ft)

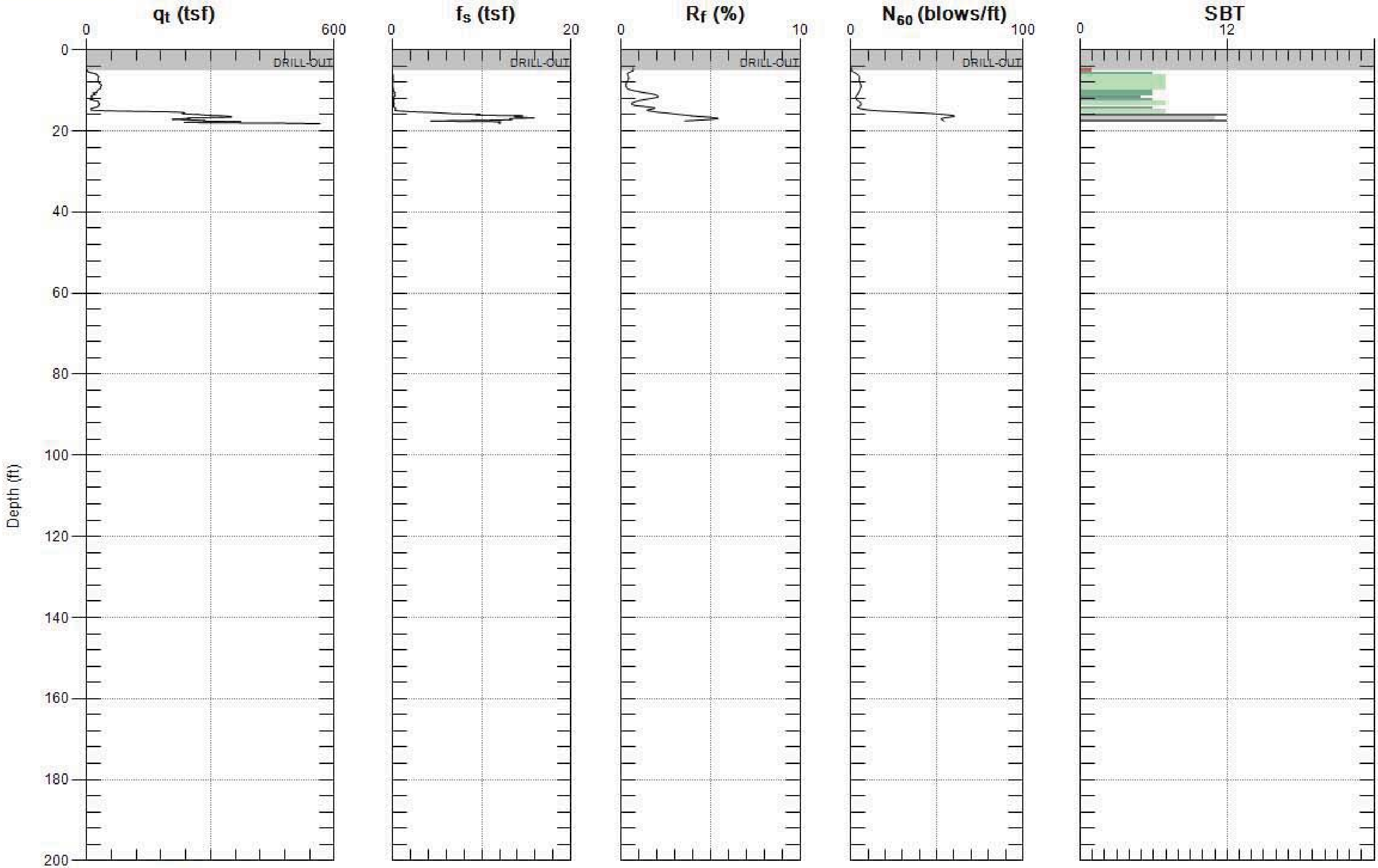
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA ENGINEERS

Site: CAMANCHE RESERVOIR **Engineer: R.HARLAN**

Sounding: MC-11 **Date: 9/17/2008 03:08**



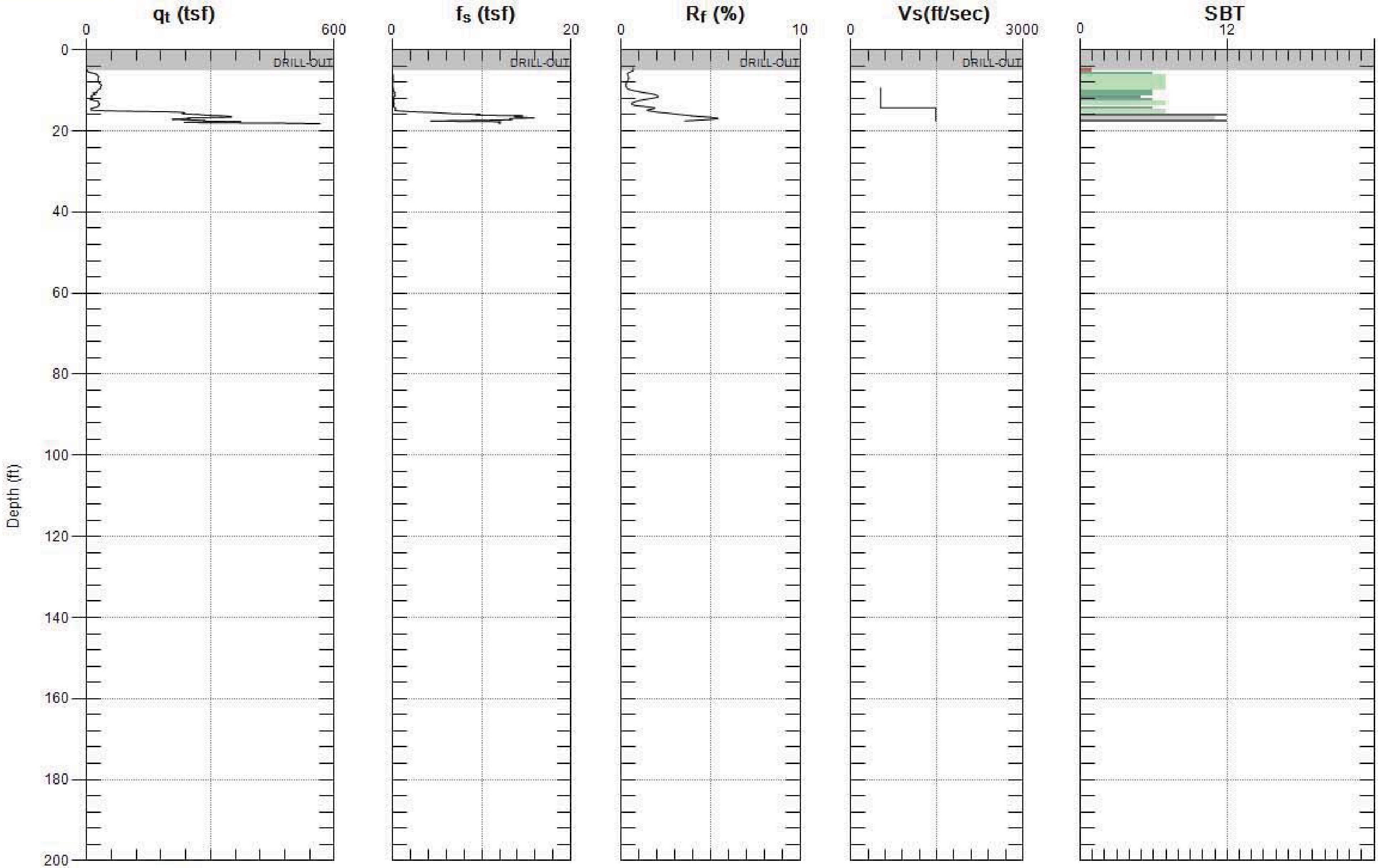
Max. Depth: 18.209 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TERRA ENGINEERS**

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-11 Date: 9/17/2008 03:08



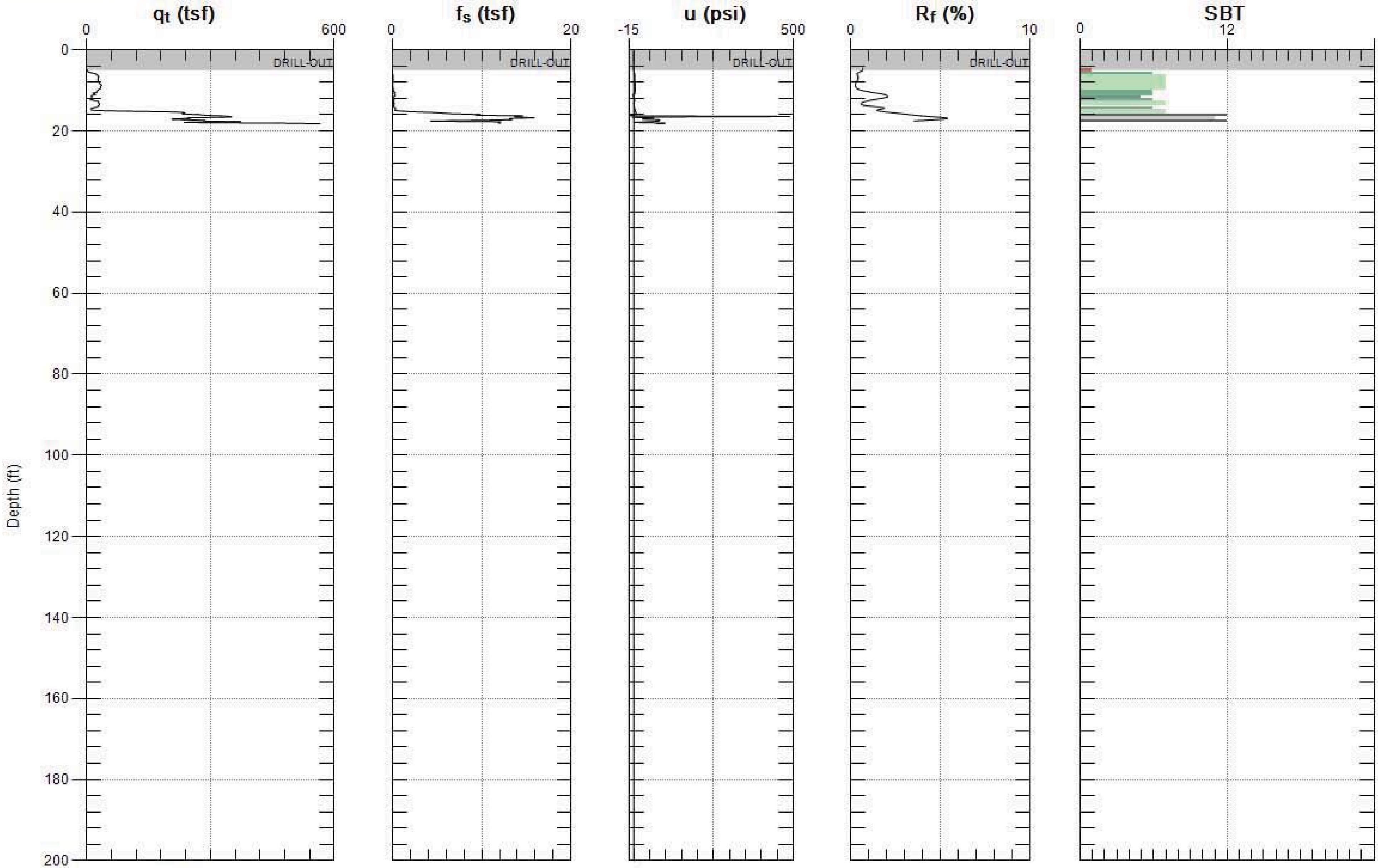
Max. Depth: 18.209 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG TERRA ENGINEERS

Site: CAMANCHE RESERVOIR Engineer: R.HARLAN

Sounding: MC-11 Date: 9/17/2008 03:08



Max. Depth: 18.209 (ft)
Avg. Interval: 0.656 (ft)

SBT: Soil Behavior Type (Robertson 1990)

APPENDIX C

TABLE C-1
SUMMARY OF SOIL LABORATORY DATA

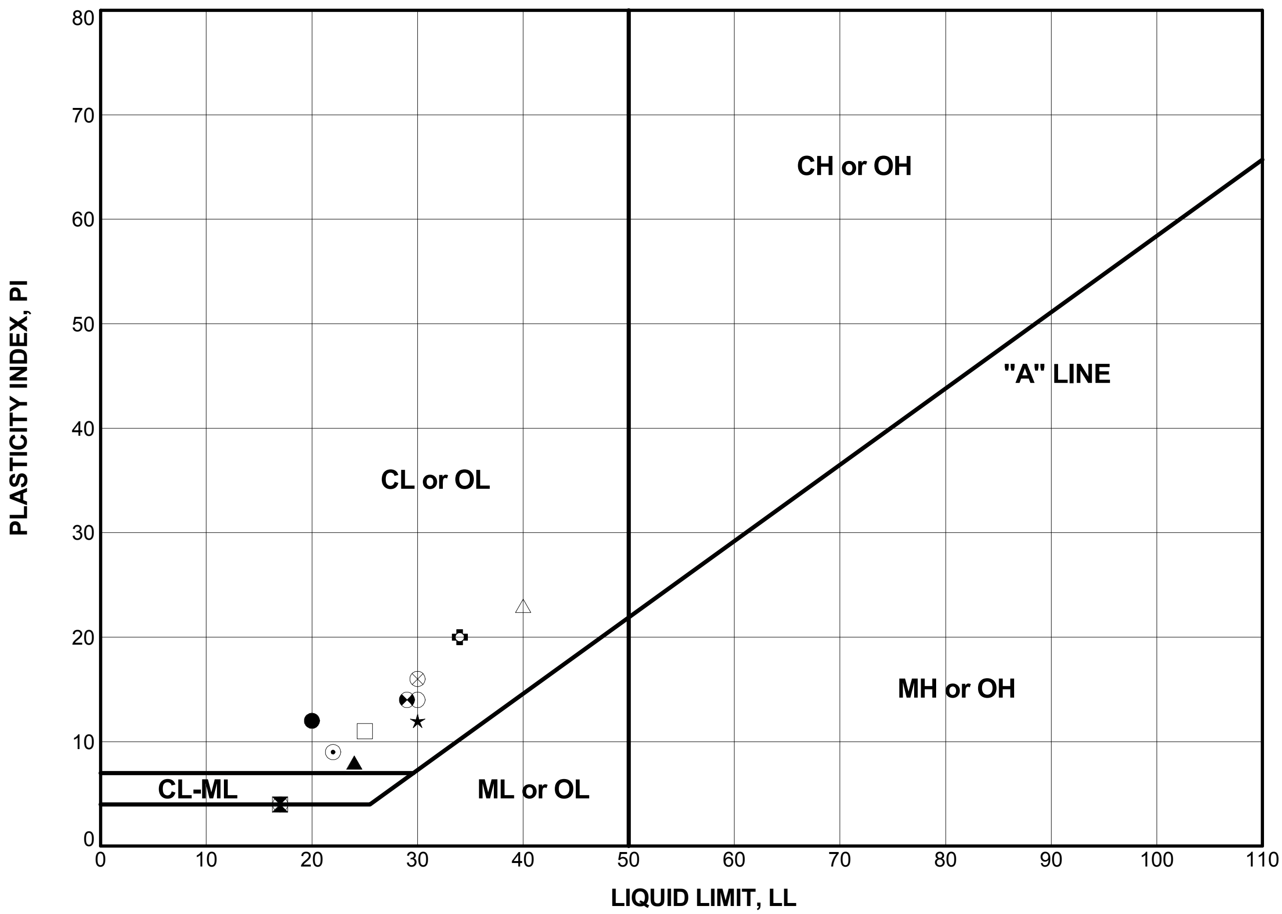
Sample Information				USCS Group Symbol	In Situ Water Content, %	In Situ Dry Unit Weight, pcf	Sieve / Hydrometer				Atterberg Limits			Other Tests
Boring Number	Sample Number	Depth, feet	Elevation, feet MSL				Gravel, %	Sand, %	<#200, %	<5μ, %	LL	PL	PI	
MB-1	SPT-1	14.5-16.0	248.0	SC					42		20	8	12	
MB-1	SPT-2	19.5-21.0	243.0	SC-SM			0	54	46	15	17	13	4	
MB-1	SPT-3	29.5-31.0	233.0	CL					56		24	16	8	
MB-1	SPT-4	39.5-41.0	223.0	CL					68		30	18	12	
MB-1	SH-4	45.6-46.1	217.4	CL	11.5	122.1	0	49	51	17	22	13	9	TX-CIU' (6.5)
MB-1	SH-5	51.3-51.8	211.7	CL	16.5						34	14	20	
MB-1	SH-6B	57.8-58.3	205.2	CL	21.6	104.9			78		30	16	14	TX-CIU' (11.5)
MB-1	SH-6A	58.3-58.8	204.7	CL	19.0	104.4	0	22	79	45	40	17	23	TX-CIU' (7.9)
MB-1	SH-6tip	58.8-59.0	204.2	CL							30	14	16	
MB-1	SPT-5	62.0-63.5	200.5	CL-ML			0	48	52	18	17	13	4	
MB-1	SH-7tip	68.0-68.5	195.0	CL	12.8						25	14	11	
MB-1	SH-8B	74.6-75.1	188.4	CL	14.3	117.7			65		29	15	14	TX-CIU' (13.0)
MB-1	SH-8A	75.1-75.6	187.9	CL	14.1	118.2	0	43	57	23	24	14	10	TX-CIU' (8.6)
MB-1	SH-9	79.0-79.5	184.0	CL	18.1	110.0			52		29	19	10	TX-CIU' (9.4
MB-1	SH-9tip	79.5-80.0	183.5	CL	21.8						27	18	9	
MB-1	SPT-6	83.0-84.5	179.5	CL					53		22	5	17	
MB-1	SH-10B	88.6-89.1	174.4	SC	15.5	121.5			27		25	13	12	TX-CIU' (14.4)
MB-1	SH-10A	89.1-89.6	173.9	SC	15.0	121.1	2	64	34	17	25	11	14	TX-CIU' (10.0)
MB-1	SPT-7	93.0-94.5	169.5	SM			0	66	33					
MB-1	SH-11B	98.4-98.9	164.6	CL	11.6	123.2			54		27	14	13	TX-CIU' (15.8)
MB-1	SH-11A	98.9-99.4	164.1	CL	10.0	123.6	1	49	50	15	25	15	10	TX-CIU' (10.8)
MB-1	SPT-11tip	99.4-99.6	163.6	CL-ML	11.9						23	16	7	
MB-1	SPT-8	103.0-104.5	159.5	SC-SM			0	50	50		21	17	4	
MB-1	SH-12	108.7-109.2	154.3	CL-ML	16.6						21	16	5	
MB-1	SPT-9	113.0-114.5	149.5	ML			1	42	57	17	NP	NP	NP	
MB-1	SH-13	119.0-119.5	144.0	ML	16.3						NP	NP	NP	
MB-1	SPT-10	123.0-124.5	139.5	CL-ML			0	34	66		20	13	7	
MB-1	SH-14B	128.2-128.7	134.8	SM	18.0	110.0			33		NP	NP	NP	TX-CIU' (15.8)
MB-1	SH-14A	128.7-129.2	134.3	SC	12.7	116.6	1	56	43	12	NP	NP	NP	TX-CIU' (12.2)
MB-1	SPT-11	133.0-134.5	129.5	CL			8	37	55	18	28	16	12	
MB-1	SH-15	138.3-138.8	124.7	CL	13.5	119.7	0	47	53	12	NP	NP	NP	TX-CIU' (13.0)
MB-1	SPT-12	143.0-144.5	119.5	SC-SM			5	46	50		21	16	5	
MB-1	SPT-13	151.0-152.5	111.5	SM			19	46	35	8	NP	NP	NP	

TABLE C-1
SUMMARY OF SOIL LABORATORY DATA

Sample Information				USCS Group Symbol	In Situ Water Content, %	In Situ Dry Unit Weight, pcf	Sieve / Hydrometer				Atterberg Limits			Other Tests
Boring Number	Sample Number	Depth, feet	Elevation, feet MSL				Gravel, %	Sand, %	<#200, %	<5μ, %	LL	PL	PI	
MB-1	SPT-14	162.0-163.5	100.5	CL-ML			5	43	53	17	24	19	5	
MB-1	SH-18	168.3-168.8	94.7	SC	14.3						27	19	8	
MB-1	SPT-15	173.0-174.5	89.5	CL					61		26	18	8	
MB-1	SH-19	178.2-178.7	84.8	CL-ML	13.3	116.7	0	42	58	17	22	15	7	TX-CIU' (15.8)
MB-1	SH-19tip	178.7-179.2	84.3	CL-ML	12.4						25	18	7	
MB-1	SPT-16	183.0-183.5	80.0	ML			1	40	59	10				
MB-1	SPT-17	188.0-189.5	74.5	SM			22	34	44	14				
MB-1	SPT-18	193.0-193.7	70.0	ML			0	22	78	10				
MB-2	SPT-1	4.5-6.0	105.0	SP-SM					10					
MB-2	SPT-2	7.0-8.5	102.5	SP					4					
MB-2	SPT-3	9.5-11.0	100.0	SP-SM					6					
MB-2	SPT-4	12.0-13.5	97.5	SP-SM					10					
MB-2	SPT-5	14.5-16.0	95.0	SP					4					
MB-2	SPT-7	19.5-21.0	90.0	SM					30		NP	NP	NP	
MB-3	SPT-1	6.0-7.5	98.0	SP-SM			1	91	8	3				
MB-3	SPT-2	7.5-9.0	96.5	SP-SM			0	90	10	3				
MB-3	SPT-3	10.0-11.5	94.0	SP-SM			9	87	5	1				
MB-3	SPT-4	12.5-14.0	91.5	SM			2	65	34	5				
MB-3	SPT-5	15.0-16.0	89.3	ML			7	34	59	7				
MB-4	SPT-1	9.5-11.0	100.0	SP-SM					6					
MB-4	SPT-2	14.0-16.3	95.1	CH					98		52	27	25	
MB-4	SH-2	18.0-18.5	92.0	SM	34.8	88.9	0	83	17	3	NP	NP	NP	
MB-4	SPT-3	19.0-21.0	90.3	SM					39		NP	NP	NP	
MB-4	SPT-4	22.0-23.5	87.5	SM					43		NP	NP	NP	
MB-5	SPT-1	21.0-22.7	93.4	SM					34		NP	NP	NP	
MB-5	SPT-2	23.5-25.0	91.0	SM					13		NP	NP	NP	
MB-5	SPT-3	26.0-27.1	88.8	SM					42		NP	NP	NP	

NOTE: The laboratory tests were performed in general accordance with the following ASTM standards:

Moisture Content - ASTM Test Method D2216
Dry Unit Weight - ASTM Test Method D2937
Particle Size Distribution Analysis by Sieving and Hydrometer - ASTM Test Method D422
Percent Passing No. 200 Sieve - ASTM Test Method D1140
Atterberg Limits - ASTM Test Method D4318
One-Dimensional Consolidation Test (CONS) - ASTM Test Method D2435
Unconfined Compression Test (UC) - ASTM Test Method D2166
Consolidated Undrained Triaxial Test with Pore Pressure Measurement (TX-CIU', Confining Pressure in ksf) - ASTM Test Method D4767



Boring Number	Sample Number	Depth (feet)	Test Symbol	Water Content (%)	LL	PL	PI	Classification
MB-1	SPT-1	14.5-16.0	●		20	8	12	Clayey Sand (SC)
MB-1	SPT-2	19.5-21.0	⧗		17	13	4	Silty, Clayey Sand (SC-SM)
MB-1	SPT-3	29.5-31.0	▲		24	16	8	Sandy Lean Clay (CL)
MB-1	SPT-4	39.5-41.0	★		30	18	12	Sandy Lean Clay (CL)
MB-1	SH-4	45.6-46.1	⊙	11.5	22	13	9	Sandy Lean Clay (CL)
MB-1	SH-5	51.3-51.8	✪	16.5	34	14	20	Sandy Lean Clay (CL)
MB-1	SH-6B	57.8-58.3	○	21.6	30	16	14	Lean Clay with Sand (CL)
MB-1	SH-6A	58.3-58.8	△	19.0	40	17	23	Lean Clay with Sand (CL)
MB-1	SH-6tip	58.8-59.0	⊗		30	14	16	Lean Clay with Sand (CL)
MB-1	SPT-5	62.0-63.5	⊕		17	13	4	Sandy Silty Clay (CL-ML)
MB-1	SH-7tip	68.0-68.5	□	12.8	25	14	11	Sandy Lean Clay (CL)
MB-1	SH-8B	74.6-75.1	⧓	14.3	29	15	14	Sandy Lean Clay (CL)

PLASTICITY CHART

CAMANCHE EMBANKMENTS SAFETY REVIEW
Northern California

Figure C-1

Report: ATTERBERG_PLOT_12 PTS; File: TGP_CAMANCHE.GPJ; 12/16/2008

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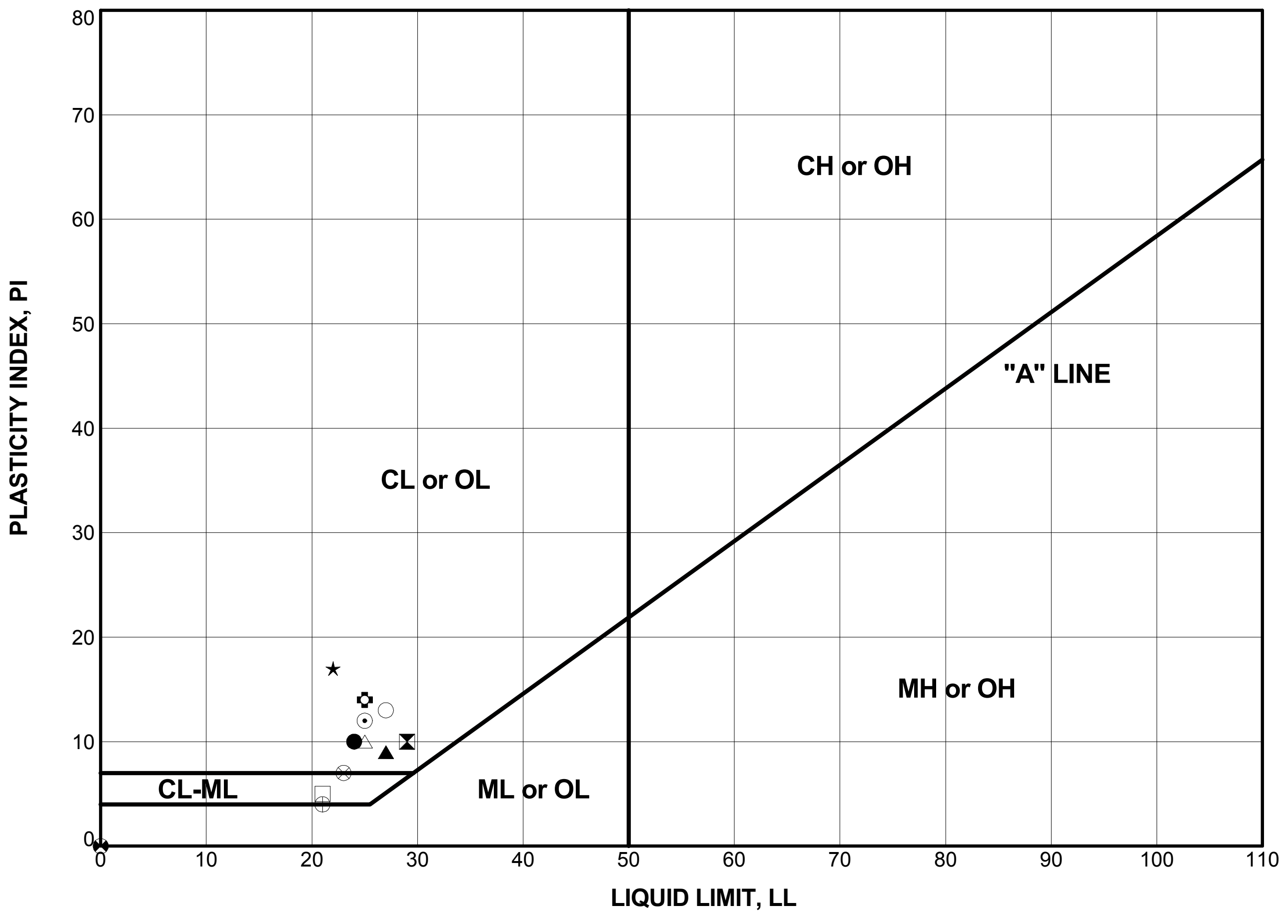

Boring Number	Sample Number	Depth (feet)	Test Symbol	Water Content (%)	LL	PL	PI	Classification
MB-1	SH-8A	75.1-75.6	●	14.1	24	14	10	Sandy Lean Clay (CL)
MB-1	SH-9	79.0-79.5	⧗	18.1	29	19	10	Sandy Lean Clay (CL)
MB-1	SH-9tip	79.5-80.0	▲	21.8	27	18	9	Sandy Lean Clay (CL)
MB-1	SPT-6	83.0-84.5	★		22	5	17	Sandy Lean Clay (CL)
MB-1	SH-10B	88.6-89.1	⊙	15.5	25	13	12	Clayey Sand (SC)
MB-1	SH-10A	89.1-89.6	✪	15.0	25	11	14	Clayey Sand (SC)
MB-1	SH-11B	98.4-98.9	○	11.6	27	14	13	Sandy Lean Clay (CL)
MB-1	SH-11A	98.9-99.4	△	10.0	25	15	10	Sandy Lean Clay (CL)
MB-1	SPT-11tip	99.4-99.6	⊗	11.9	23	16	7	Sandy Silty Clay (CL-ML)
MB-1	SPT-8	103.0-104.5	⊕		21	17	4	Silty, Clayey Sand (SC-SM)
MB-1	SH-12	108.7-109.2	□	16.6	21	16	5	Sandy Silty Clay (CL-ML)
MB-1	SPT-9	113.0-114.5	⧓		NP	NP	NP	Sandy Silt (ML)

PLASTICITY CHART

CAMANCHE EMBANKMENTS SAFETY REVIEW
Northern California

Figure C-2

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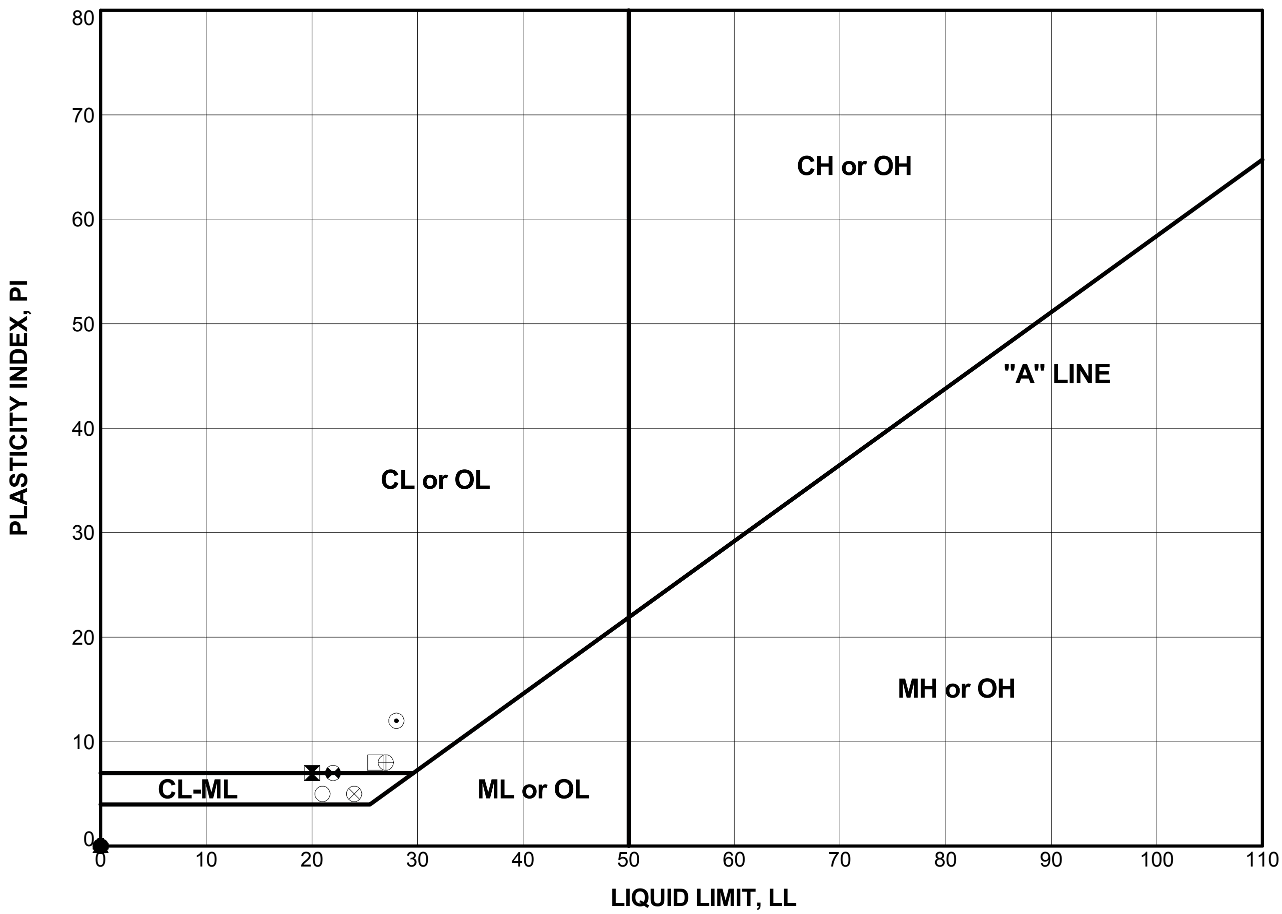

Boring Number	Sample Number	Depth (feet)	Test Symbol	Water Content (%)	LL	PL	PI	Classification
MB-1	SH-13	119.0-119.5	●	16.3	NP	NP	NP	Sandy Silt (ML)
MB-1	SPT-10	123.0-124.5	⧗		20	13	7	Sandy Silty Clay (CL-ML)
MB-1	SH-14B	128.2-128.7	▲	18.0	NP	NP	NP	Silty Sand (SM)
MB-1	SH-14A	128.7-129.2	★	12.7	NP	NP	NP	Silty Sand (SM)
MB-1	SPT-11	133.0-134.5	⊙		28	16	12	Sandy Lean Clay (CL)
MB-1	SH-15	138.3-138.8	✿	13.5	NP	NP	NP	Sandy Lean Clay (CL)
MB-1	SPT-12	143.0-144.5	○		21	16	5	Silty, Clayey Sand (SC-SM)
MB-1	SPT-13	151.0-152.5	△		NP	NP	NP	Silty Sand with Gravel (SM)
MB-1	SPT-14	162.0-163.5	⊗		24	19	5	Sandy Silty Clay (CL-ML)
MB-1	SH-18	168.3-168.8	⊕	14.3	27	19	8	Clayey Sand (SC)
MB-1	SPT-15	173.0-174.5	□		26	18	8	Sandy Lean Clay (CL)
MB-1	SH-19	178.2-178.7	⧓	13.3	22	15	7	Sandy Silty Clay (CL-ML)

PLASTICITY CHART

CAMANCHE EMBANKMENTS SAFETY REVIEW
Northern California

Figure C-3

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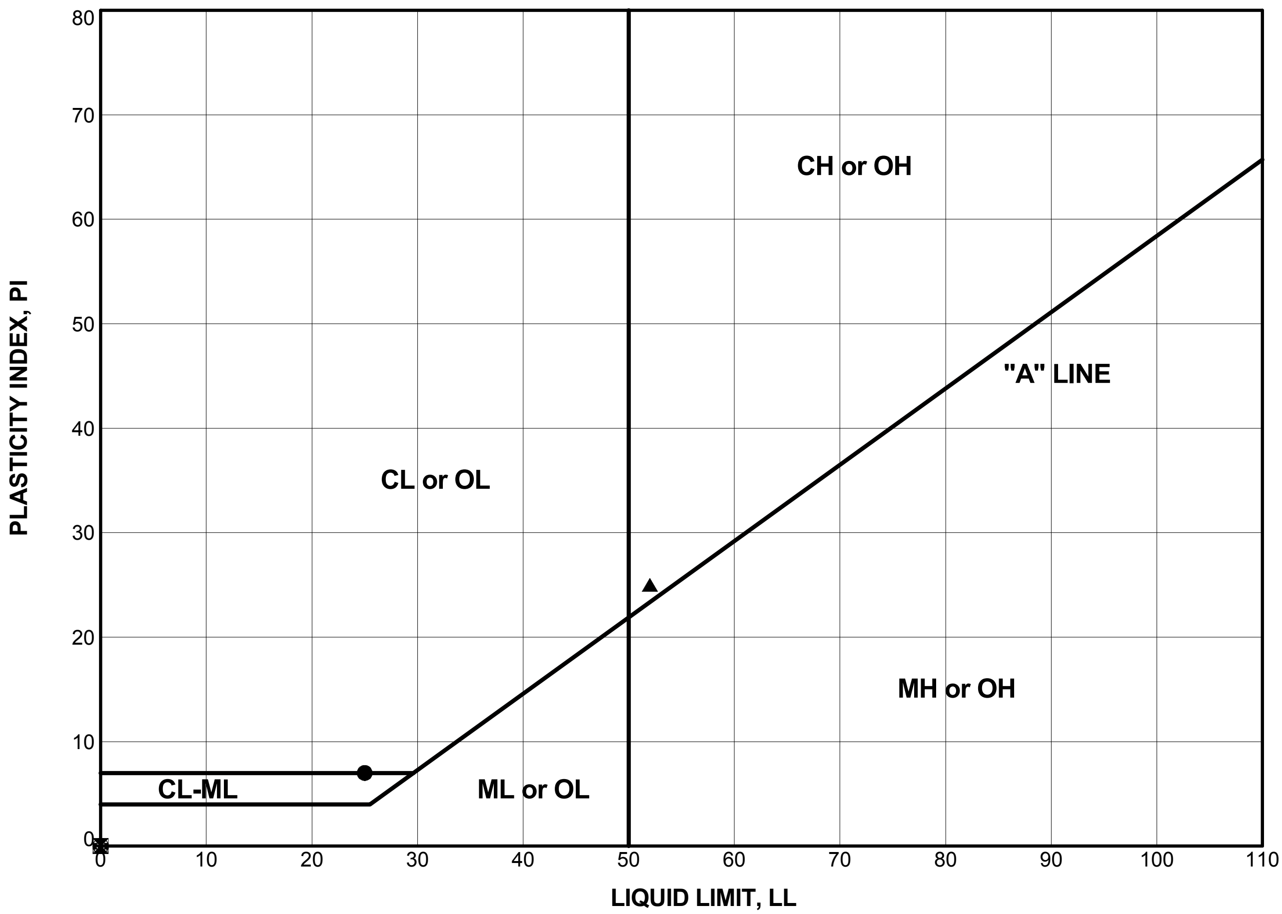

Boring Number	Sample Number	Depth (feet)	Test Symbol	Water Content (%)	LL	PL	PI	Classification
MB-1	SH-19tip	178.7-179.2	●	12.4	25	18	7	Sandy Silty Clay (CL-ML)
MB-2	SPT-7	19.5-21.0	⧗		NP	NP	NP	Silty Sand (SM)
MB-4	SPT-2	14.0-16.3	▲		52	27	25	Fat Clay (CH)
MB-4	SH-2	18.0-18.5	★	34.8	NP	NP	NP	Silty Sand (SM)
MB-4	SPT-3	19.0-21.0	⊙		NP	NP	NP	Silty Sand (SM)
MB-4	SPT-4	22.0-23.5	✿		NP	NP	NP	Silty Sand (SM)
MB-5	SPT-1	21.0-22.7	○		NP	NP	NP	Silty Sand (SM)
MB-5	SPT-2	23.5-25.0	△		NP	NP	NP	Silty Sand with Gravel (SM)
MB-5	SPT-3	26.0-27.1	⊗		NP	NP	NP	Silty Sand (SM)

PLASTICITY CHART

CAMANCHE EMBANKMENTS SAFETY REVIEW
Northern California

Figure C-4

TERRA / GeoPentech
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Enclosure 1

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by: KK	Date: 10-14-08
Project No.:	N/A	Input Data by: KM	Date: 10-23-08
Boring No.:	MB-1	Reviewed by: AP	Date: 10-23-08
Sample No.:	SH-4	Sample Description: Sandy Clay	
Depth(ft):	44.5-46.3		
Test Condition:	Shelby Tube	Confining Pressure = 6.5 ksf	

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	5.750	5.750	5.750	Avg. =	5.750

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.447
Moisture Content (%)	11.48	13.18
Wet Weight (gms)	486.20	1499.73
Dry Weight (gms)	456.26	1343.01
Container Weight (gms)	195.37	154.15
Density and Saturation		
Wet Weight (gms)	1334.09	
Container Weight (gms)	0.00	
Wet Density (pcf)	136.2	
Dry Density (pcf)	122.1	
Initial Void Ratio	0.379	
% Saturation	81.7	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	95	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	85.0	Initial Burette Ht.(cm)=	52.4
Back Pressure(psi) =	40.0	Final Burette Ht.(cm)=	38.6
Eff. Consol. Stress (psi) =	45.0	Final Height (in)=	5.659
Induced OCR =	1.0	Initial Volume (cu.in)=	37.328
Change in Ht. of Specimen (in) =	0.0910	Final Volume (cu.in) =	36.486

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	30.70
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	10.48
Failure Criteria:		Eff. Major Principal stress (ksf) =	41.19
Condition at which maximum deviator stress occurs		Axial Strain (%) =	14.14

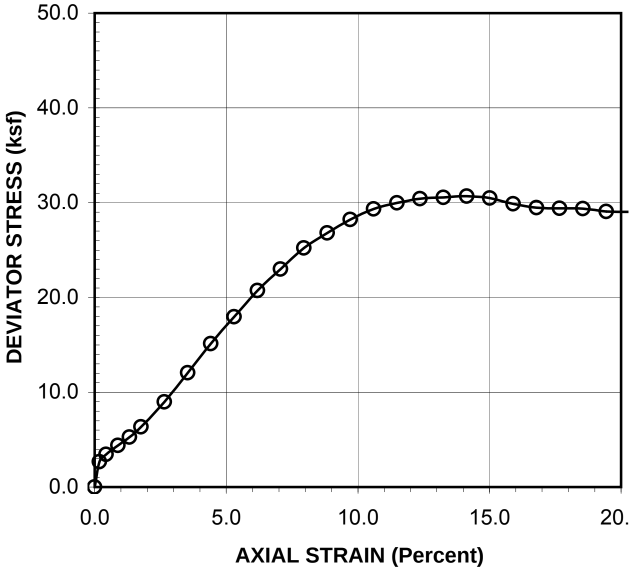
CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

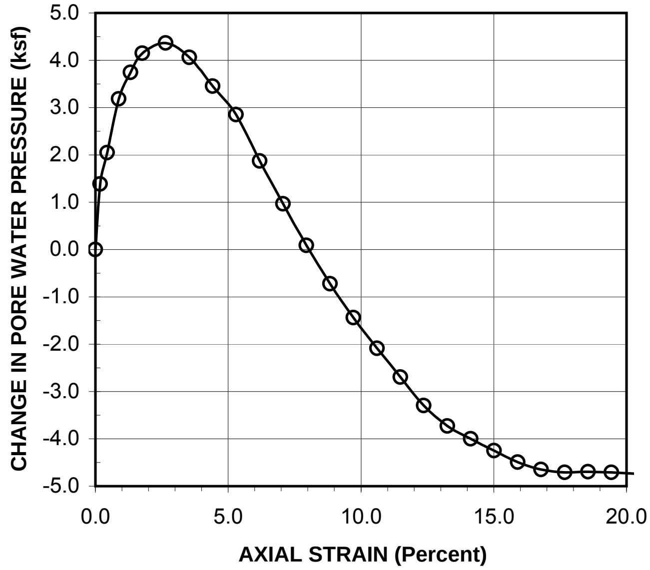
Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	85.0	psi
Project No:	N/A	Back Pressure :	40.0	psi
Boring No.:	MB-1	Consolidation Pressure :	45.0	psi
Depth(ft):	44.5-46.3	Initial Sample Height:	5.750	in
Sample No.:	SH-4	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.659	in
Sample Description:	Sandy Clay	Final Sample Area (A)*:	6.447	sq. in.
		Induced OCR=	1.0	

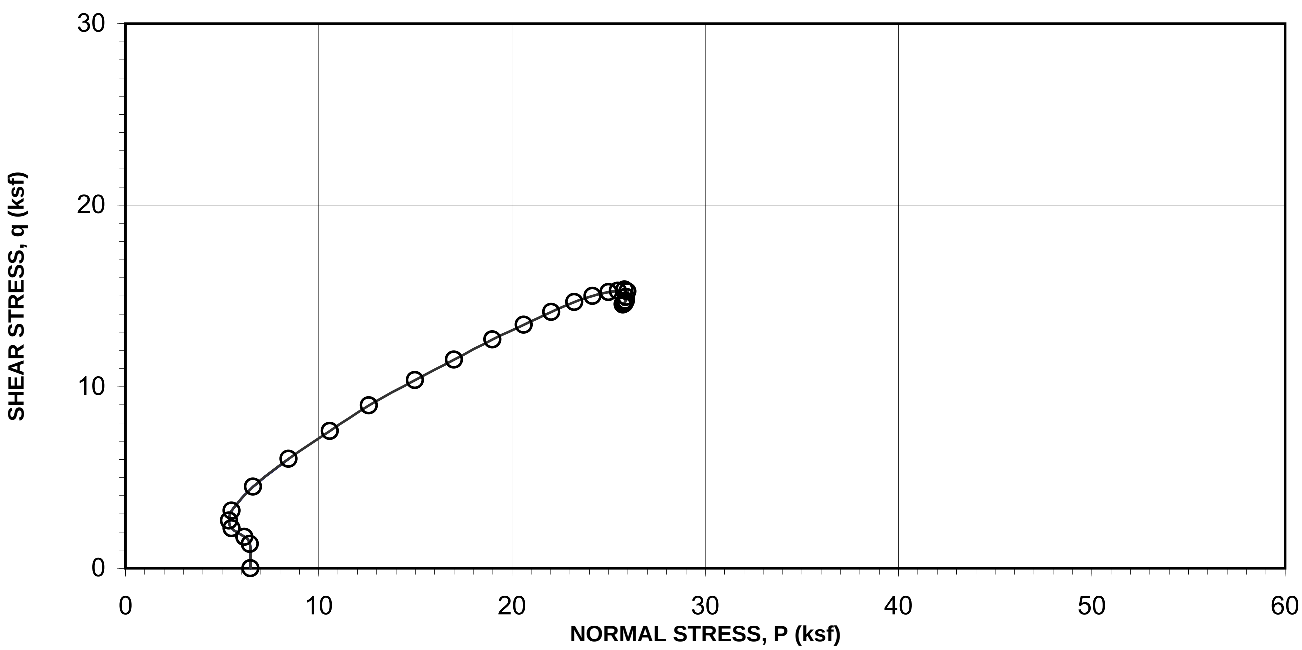
Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
85.0	0	0.000	40.0	0.00	0.00	0.00	0.00	6.48
85.0	120	0.010	49.6	2.68	0.18	1.38	1.34	6.44
85.0	155	0.025	54.2	3.45	0.44	2.04	1.72	6.16
85.0	199	0.050	62.1	4.41	0.88	3.18	2.20	5.50
85.0	239	0.075	66.0	5.27	1.33	3.74	2.63	5.37
85.0	289	0.100	68.8	6.34	1.77	4.15	3.17	5.50
85.0	413	0.150	70.3	8.98	2.65	4.36	4.49	6.61
85.0	559	0.200	68.2	12.04	3.53	4.06	6.02	8.44
85.0	708	0.250	64.0	15.11	4.42	3.46	7.56	10.58
85.0	849	0.300	59.8	17.96	5.30	2.85	8.98	12.61
85.0	990	0.350	53.0	20.74	6.18	1.87	10.37	14.98
85.0	1107	0.400	46.7	22.98	7.07	0.96	11.49	17.00
85.0	1226	0.450	40.6	25.20	7.95	0.09	12.60	19.00
85.0	1317	0.500	35.0	26.82	8.84	-0.72	13.41	20.61
85.0	1400	0.550	30.0	28.23	9.72	-1.44	14.11	22.03
85.0	1469	0.600	25.5	29.33	10.60	-2.09	14.67	23.23
85.0	1517	0.650	21.3	29.99	11.49	-2.69	14.99	24.17
85.0	1555	0.700	17.1	30.43	12.37	-3.30	15.22	24.99
85.0	1577	0.750	14.1	30.55	13.25	-3.73	15.28	25.49
85.0	1601	0.800	12.2	30.70	14.14	-4.00	15.35	25.83
85.0	1607	0.850	10.5	30.50	15.02	-4.25	15.25	25.98
85.0	1591	0.900	8.8	29.88	15.90	-4.49	14.94	25.91
85.0	1585	0.950	7.7	29.46	16.79	-4.65	14.73	25.86
85.0	1600	1.000	7.3	29.42	17.67	-4.71	14.71	25.90
85.0	1615	1.050	7.4	29.38	18.55	-4.69	14.69	25.86
85.0	1616	1.100	7.3	29.08	19.44	-4.71	14.54	25.73
85.0	1632	1.150	7.1	29.04	20.32	-4.74	14.52	25.74
85.0	1656	1.200	6.8	29.14	21.21	-4.78	14.57	25.83

AP Engineering & Testing, Inc.





LEGEND: CONFINING PRESSURE= ○ 6.5 KSF

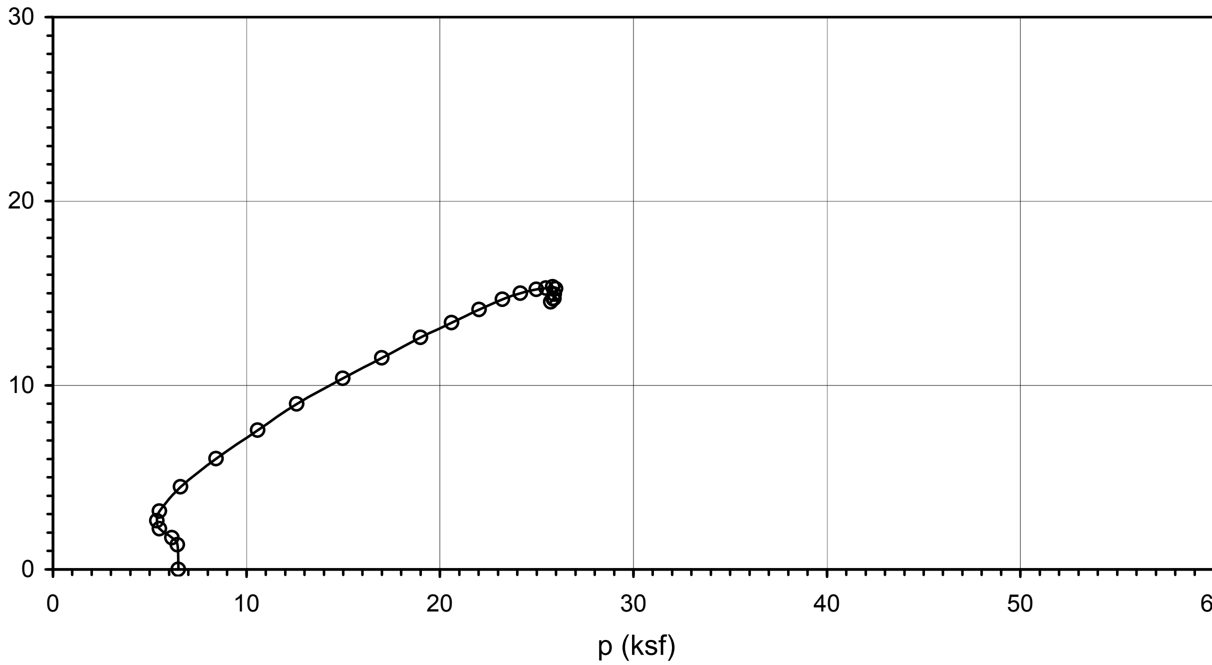


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	122.1
Sample No.:	SH-4	Initial Moisture Content (%):	11.5
Depth (ft):	44.5-46.3	Confining Pressure:	6.5 ksf

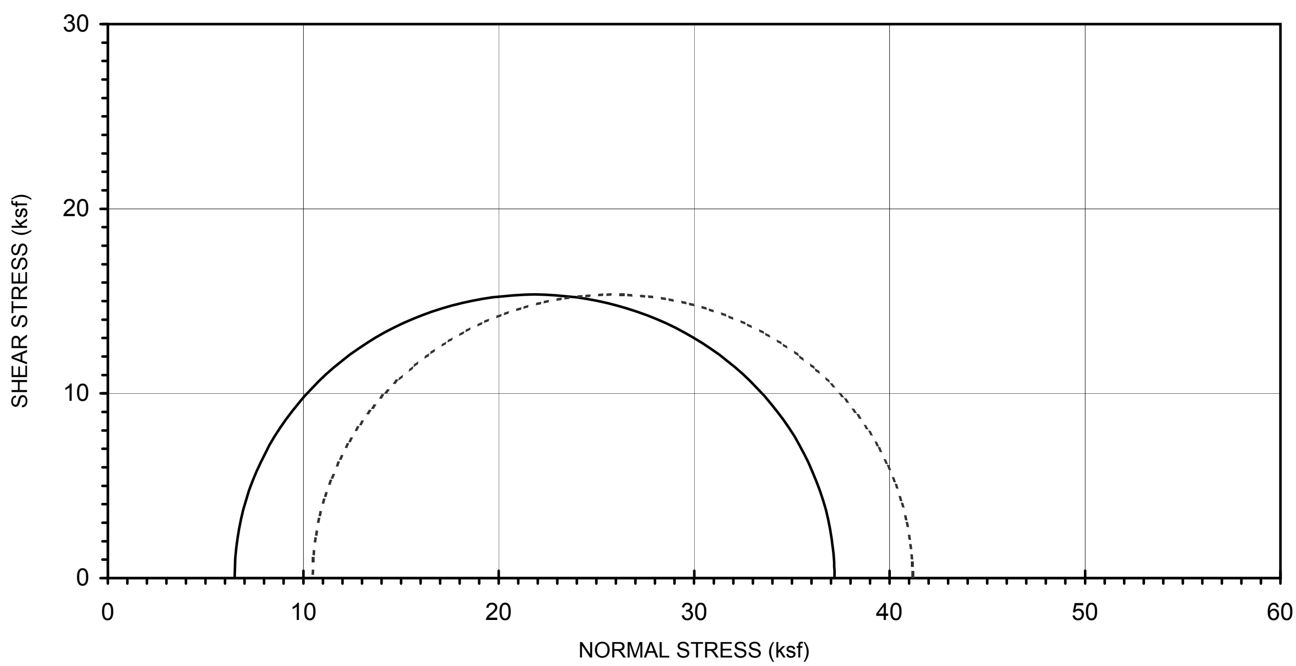
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 6.5 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	122.1
Sample No.:	SH-4	Initial Moisture Content (%):	11.5
Depth (ft):	44.5-46.3	Confining Pressure:	6.5 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by: KK	Date: 10-28-08
Project No.:	N/A	Input Data by: KM	Date: 11-06-08
Boring No.:	MB-1	Reviewed by: AP	Date: 11-06-08
Sample No.:	SH-6 (B)	Sample Description: Lean Clay with sand	
Depth(ft):	57-59		
Test Condition:	Shelby Tube	Confining Pressure = 11.5 ksf	

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	6.000	6.000	6.000	Avg. =	6.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.304
Moisture Content (%)	21.58	18.66
Wet Weight (gms)	171.85	1477.14
Dry Weight (gms)	150.08	1274.27
Container Weight (gms)	49.20	186.89
Density and Saturation		
Wet Weight (gms)	1304.34	
Container Weight (gms)	0.00	
Wet Density (pcf)	127.6	
Dry Density (pcf)	104.9	
Initial Void Ratio	0.606	
% Saturation	96.2	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	95	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	110.0	Initial Burette Ht.(cm)=	55.2
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	25.2
Eff. Consol. Stress (psi) =	80.0	Final Height (in)=	5.888
Induced OCR =	1.0	Initial Volume (cu.in)=	38.951
Change in Ht. of Specimen (in) =	0.1120	Final Volume (cu.in) =	37.120

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	10.79
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	5.47
Failure Criteria:		Eff. Major Principal stress (ksf) =	16.26
Condition at which maximum deviator stress occurs		Axial Strain (%) =	3.40

AP Engineering & Testing, Inc.

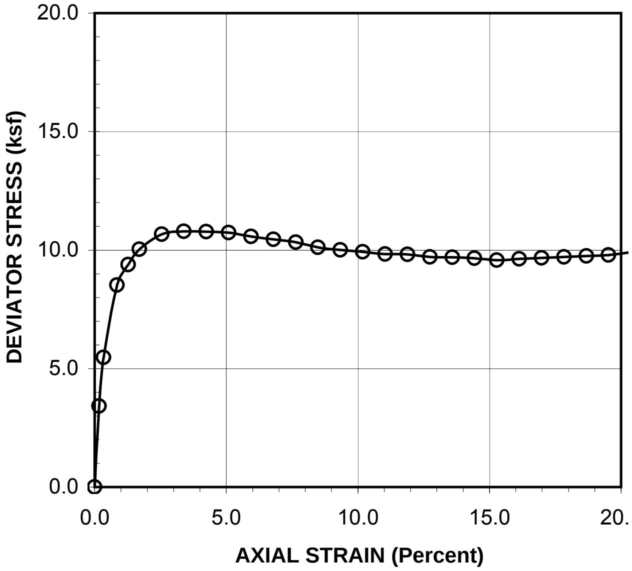
CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

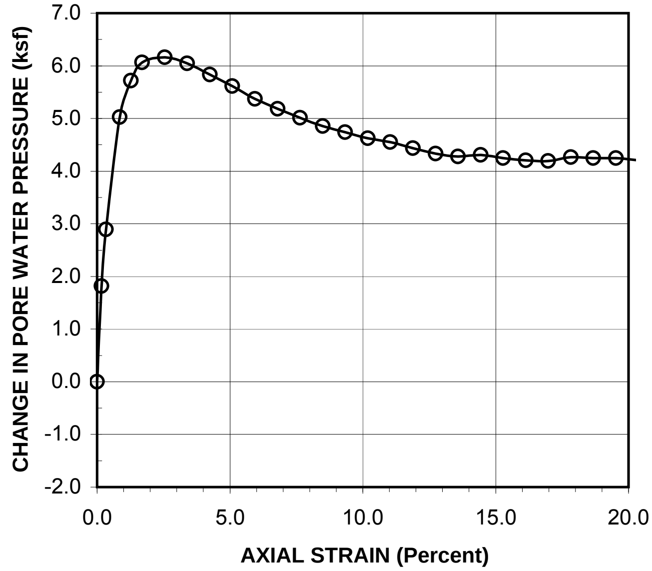
Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	110.0	psi
Project No:	N/A	Back Pressure :	30.0	psi
Boring No.:	MB-1	Consolidation Pressure :	80.0	psi
Depth(ft):	57-59	Initial Sample Height:	6.000	in
Sample No.:	SH-6 (B)	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.888	in
Sample Description:	Lean Clay with sand	Final Sample Area (A)*:	6.304	sq. in.
		Induced OCR=	1.0	

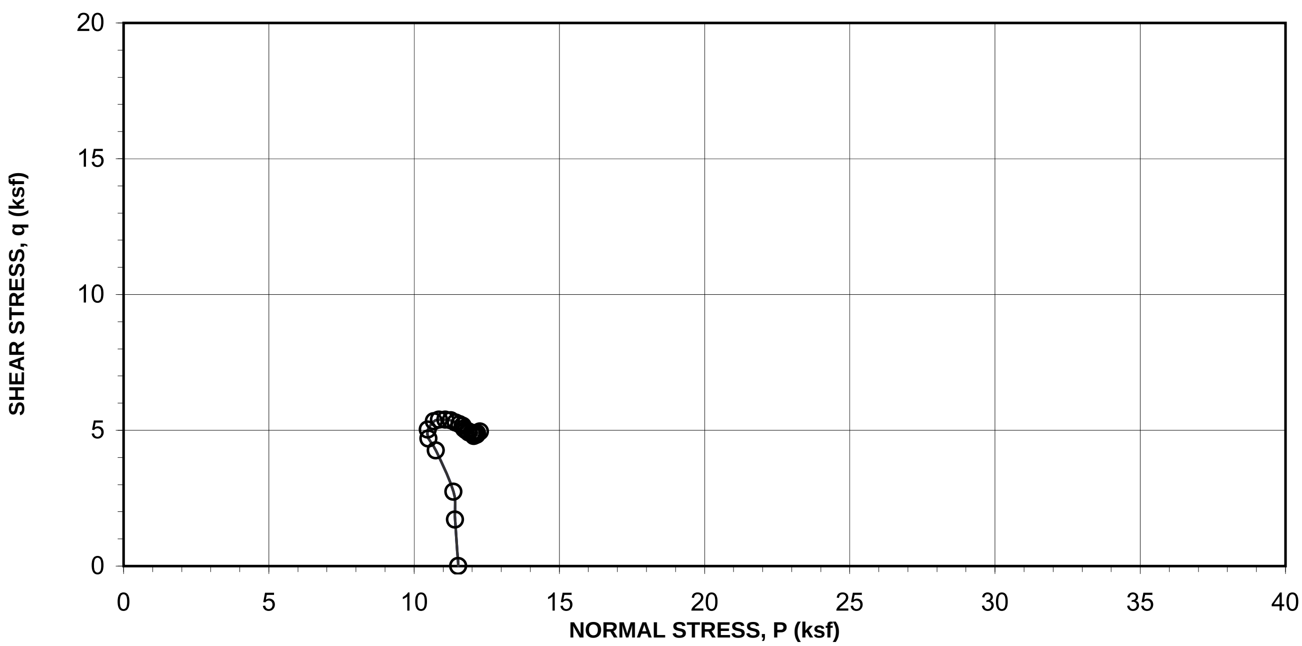
Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
110.0	0	0.000	30.0	0.00	0.00	0.00	0.00	11.52
110.0	150	0.010	42.6	3.42	0.17	1.81	1.71	11.42
110.0	240	0.020	50.1	5.46	0.34	2.89	2.73	11.36
110.0	376	0.050	64.9	8.52	0.85	5.03	4.26	10.75
110.0	416	0.075	69.7	9.38	1.27	5.72	4.69	10.49
110.0	447	0.100	72.1	10.04	1.70	6.06	5.02	10.48
110.0	479	0.150	72.8	10.66	2.55	6.16	5.33	10.69
110.0	489	0.200	72.0	10.79	3.40	6.05	5.39	10.87
110.0	493	0.250	70.5	10.78	4.25	5.83	5.39	11.08
110.0	495	0.300	69.0	10.73	5.10	5.62	5.37	11.27
110.0	492	0.350	67.3	10.57	5.94	5.37	5.28	11.43
110.0	491	0.400	66.0	10.45	6.79	5.18	5.23	11.56
110.0	490	0.450	64.8	10.34	7.64	5.01	5.17	11.68
110.0	484	0.500	63.7	10.12	8.49	4.85	5.06	11.73
110.0	483	0.550	62.9	10.00	9.34	4.74	5.00	11.78
110.0	484	0.600	62.1	9.93	10.19	4.62	4.96	11.86
110.0	484	0.650	61.6	9.83	11.04	4.55	4.92	11.89
110.0	488	0.700	60.8	9.82	11.89	4.44	4.91	12.00
110.0	487	0.750	60.1	9.71	12.74	4.33	4.85	12.04
110.0	491	0.800	59.7	9.69	13.59	4.28	4.85	12.09
110.0	494	0.850	59.9	9.65	14.44	4.31	4.83	12.04
110.0	495	0.900	59.5	9.58	15.29	4.25	4.79	12.06
110.0	503	0.950	59.2	9.64	16.13	4.20	4.82	12.13
110.0	510	1.000	59.1	9.67	16.98	4.19	4.84	12.16
110.0	517	1.050	59.6	9.70	17.83	4.26	4.85	12.11
110.0	525	1.100	59.5	9.75	18.68	4.25	4.88	12.15
110.0	533	1.150	59.5	9.80	19.53	4.25	4.90	12.17
110.0	545	1.200	59.2	9.91	20.38	4.20	4.96	12.27

AP Engineering & Testing, Inc.





LEGEND: CONFINING PRESSURE= ○ 11.5 KSF

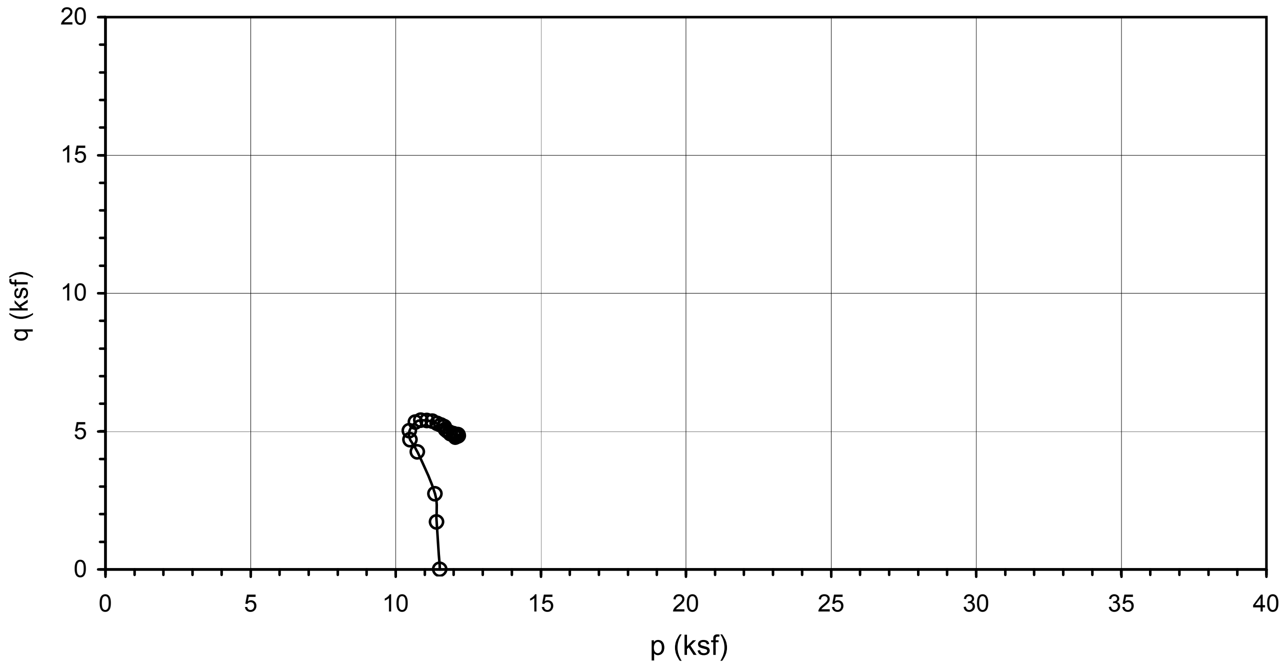


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Lean Clay with sand
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	104.9
Sample No.:	SH-6 (B)	Initial Moisture Content (%):	21.6
Depth (ft):	57-59	Confining Pressure:	11.5 ksf

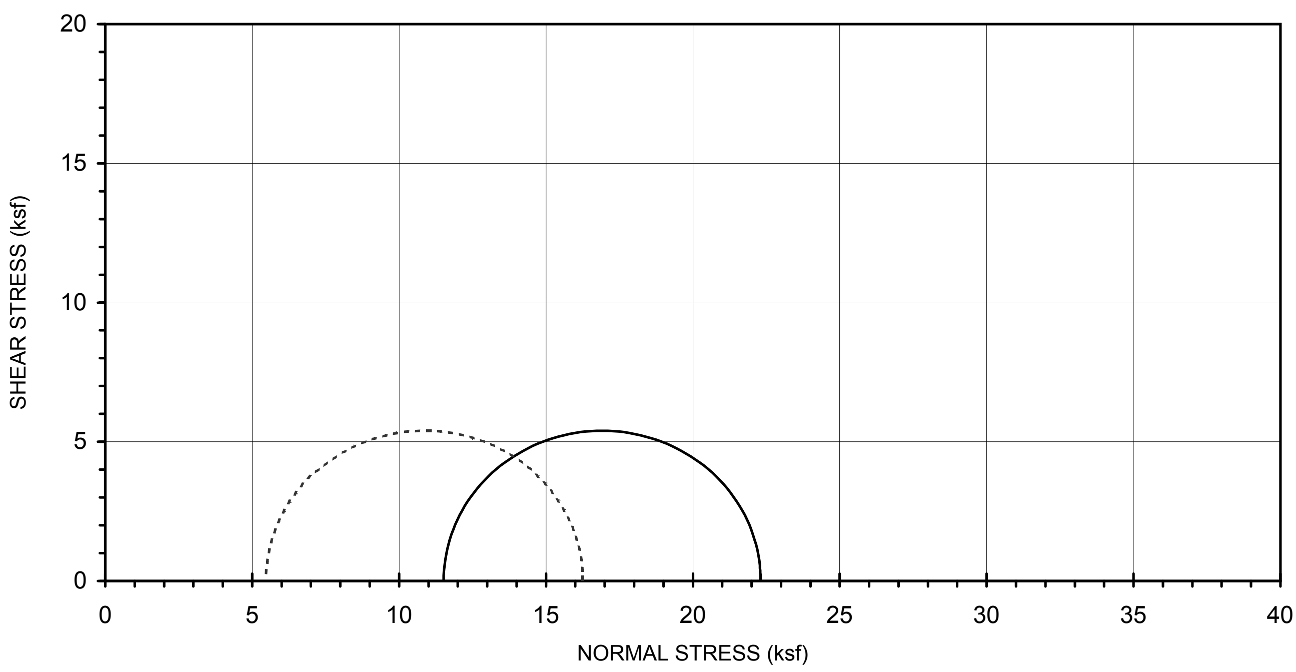
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 11.5 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Lean Clay with sand
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	104.9
Sample No.:	SH-6 (B)	Initial Moisture Content (%):	21.6
Depth (ft):	57-59	Confining Pressure:	11.5 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by: KK	Date: 10-22-08
Project No.:	N/A	Input Data by: KM	Date: 10-27-08
Boring No.:	MB-1	Reviewed by: AP	Date: 10-27-08
Sample No.:	SH-6	Sample Description: Lean Clay	
Depth(ft):	57-59		
Test Condition:	Shelby Tube	Confining Pressure = 7.9 ksf	

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	6.000	6.000	6.000	Avg. =	6.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.349
Moisture Content (%)	19.00	22.39
Wet Weight (gms)	86.22	1462.52
Dry Weight (gms)	80.44	1230.68
Container Weight (gms)	50.02	195.31
Density and Saturation		
Wet Weight (gms)	1270.32	
Container Weight (gms)	0.00	
Wet Density (pcf)	124.2	
Dry Density (pcf)	104.4	
Initial Void Ratio	0.614	
% Saturation	83.6	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	94	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	85.0	Initial Burette Ht.(cm)=	40.6
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	14.6
Eff. Consol. Stress (psi) =	55.0	Final Height (in)=	5.885
Induced OCR =	1.0	Initial Volume (cu.in)=	38.951
Change in Ht. of Specimen (in) =	0.1150	Final Volume (cu.in) =	37.365

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	8.96
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	3.90
Failure Criteria:		Eff. Major Principal stress (ksf) =	12.87
Condition at which maximum deviator stress occurs		Axial Strain (%) =	6.80

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	85.0	psi
Project No:	N/A	Back Pressure :	30.0	psi
Boring No.:	MB-1	Consolidation Pressure :	55.0	psi
Depth(ft):	57-59	Initial Sample Height:	6.000	in
Sample No.:	SH-6	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.885	in
Sample Description:	Lean Clay	Final Sample Area (A)*:	6.349	sq. in.
		Induced OCR=	1.0	

Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
85.0	0	0.000	30.0	0.00	0.00	0.00	0.00	7.92
85.0	147	0.010	40.3	3.33	0.17	1.48	1.66	8.10
85.0	204	0.020	45.3	4.61	0.34	2.20	2.31	8.02
85.0	292	0.050	53.1	6.57	0.85	3.33	3.28	7.88
85.0	331	0.075	56.0	7.41	1.27	3.74	3.71	7.88
85.0	356	0.100	57.6	7.94	1.70	3.97	3.97	7.91
85.0	381	0.150	58.6	8.42	2.55	4.12	4.21	8.01
85.0	396	0.200	58.8	8.68	3.40	4.15	4.34	8.11
85.0	407	0.250	58.5	8.84	4.25	4.10	4.42	8.24
85.0	413	0.300	58.1	8.89	5.10	4.05	4.44	8.32
85.0	418	0.350	58.5	8.92	5.95	4.10	4.46	8.28
85.0	424	0.400	57.9	8.96	6.80	4.02	4.48	8.38
85.0	427	0.450	56.9	8.94	7.65	3.87	4.47	8.52
85.0	429	0.500	56.3	8.90	8.50	3.79	4.45	8.58
85.0	433	0.550	55.5	8.90	9.35	3.67	4.45	8.70
85.0	437	0.600	54.8	8.90	10.20	3.57	4.45	8.80
85.0	439	0.650	54.2	8.86	11.05	3.48	4.43	8.86
85.0	444	0.700	54.2	8.87	11.89	3.48	4.44	8.87
85.0	447	0.750	53.7	8.85	12.74	3.41	4.42	8.93
85.0	449	0.800	52.7	8.80	13.59	3.27	4.40	9.05
85.0	452	0.850	52.6	8.77	14.44	3.25	4.39	9.05
85.0	456	0.900	51.9	8.76	15.29	3.15	4.38	9.15
85.0	458	0.950	51.1	8.71	16.14	3.04	4.36	9.24
85.0	463	1.000	50.7	8.72	16.99	2.98	4.36	9.30
85.0	467	1.050	50.6	8.70	17.84	2.97	4.35	9.30
85.0	470	1.100	49.6	8.67	18.69	2.82	4.33	9.43
85.0	478	1.150	48.8	8.72	19.54	2.71	4.36	9.57
85.0	481	1.200	48.6	8.68	20.39	2.68	4.34	9.58

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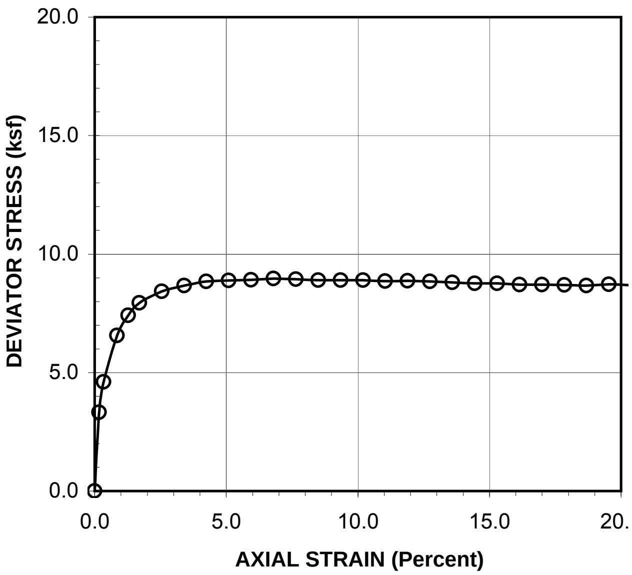
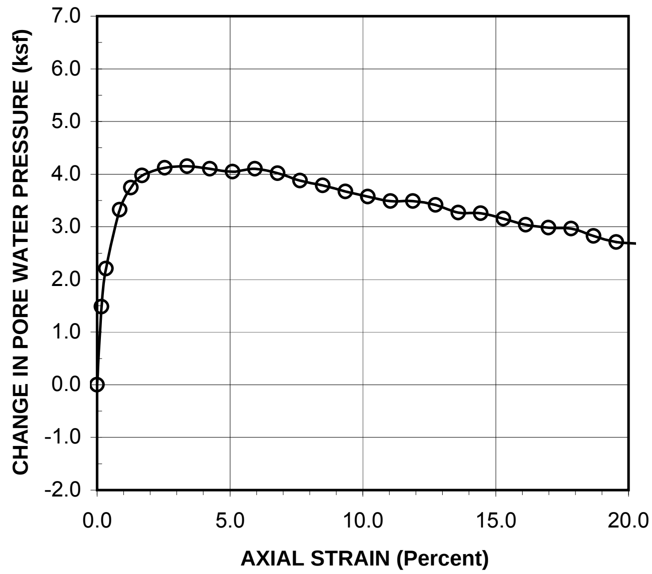
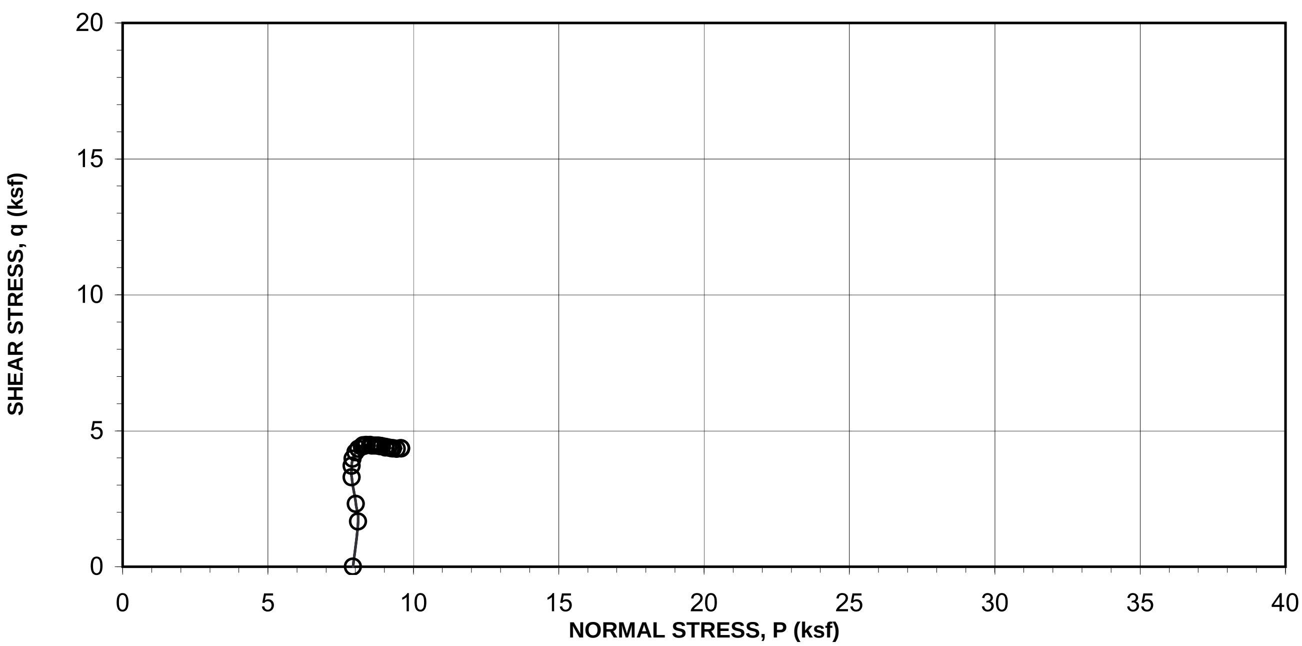




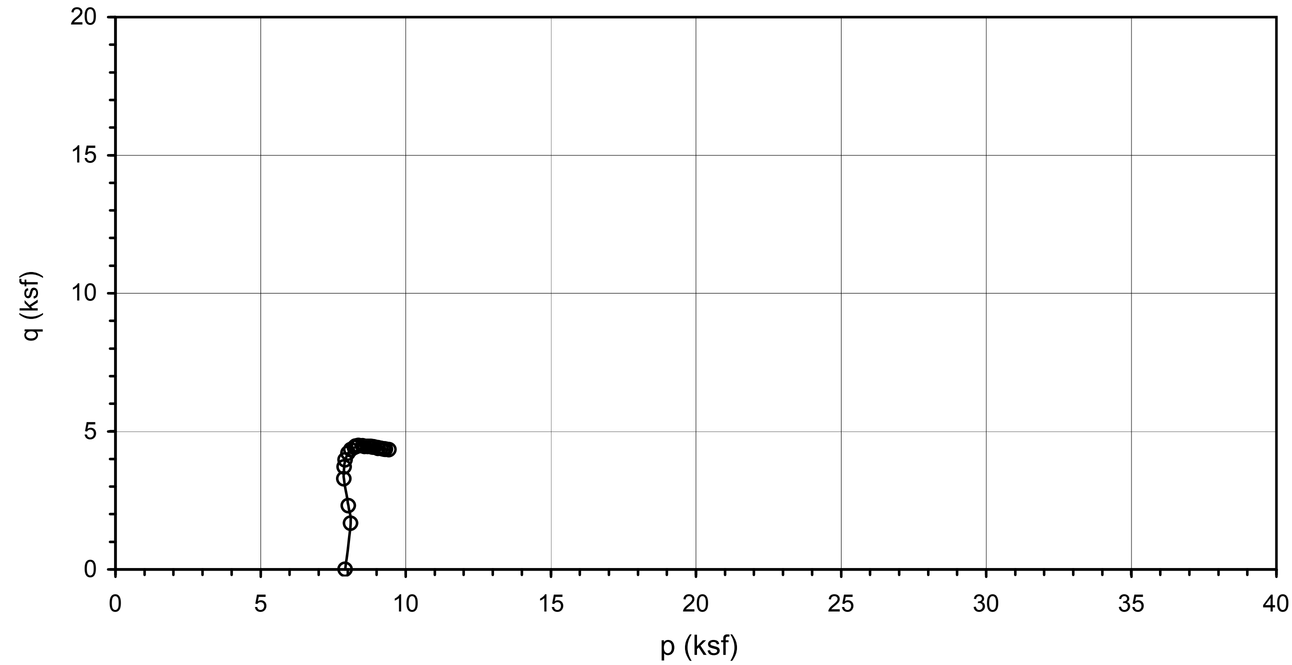
LEGEND: CONFINING PRESSURE= ○ 7.9 KSF



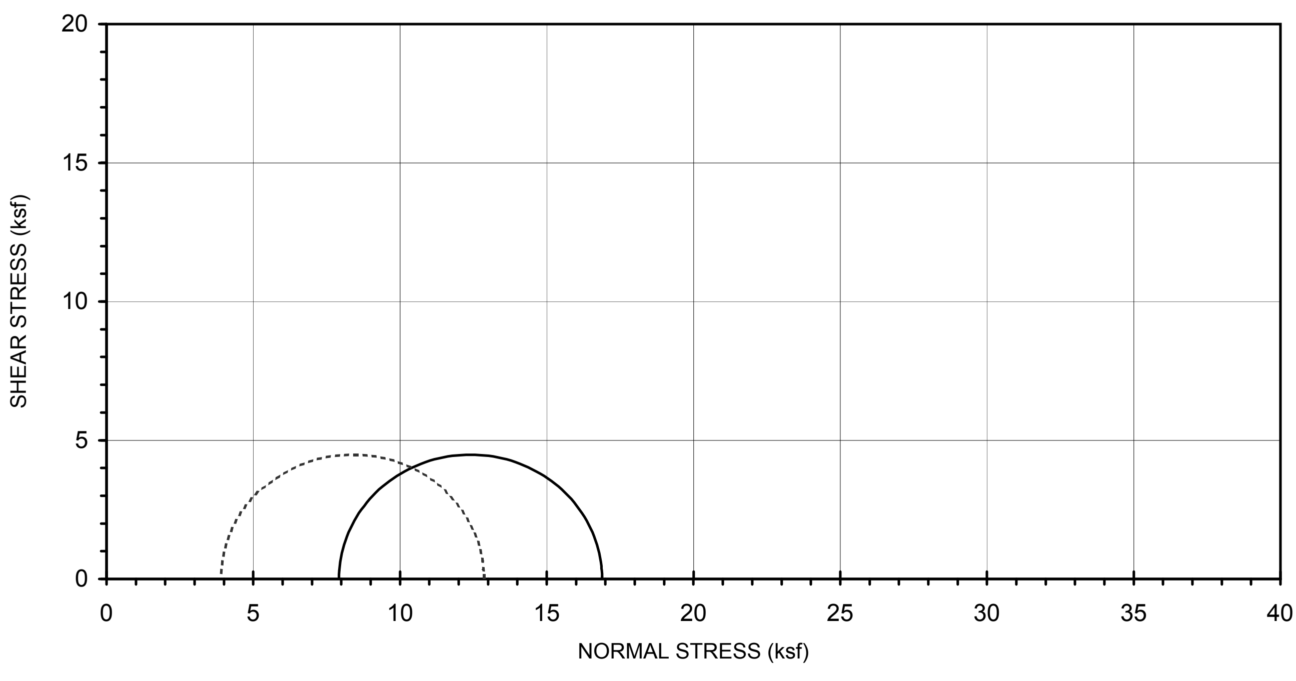
Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Lean Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	104.4
Sample No.:	SH-6	Initial Moisture Content (%):	19.0
Depth (ft):	57-59	Confining Pressure:	7.9 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT
ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 7.9 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Lean Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	104.4
Sample No.:	SH-6	Initial Moisture Content (%):	19.0
Depth (ft):	57-59	Confining Pressure:	7.9 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by:	KK	Date:	10-28-08
Project No.:	N/A	Input Data by:	KM	Date:	11-06-08
Boring No.:	MB-1	Reviewed by:	AP	Date:	11-06-08
Sample No.:	SH-8 (B)	Sample Description:	Sandy Clay		
Depth(ft):	74-75.8				
Test Condition:	Shelby Tube	Confining Pressure = 13.0 ksf			

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	6.000	6.000	6.000	Avg. =	6.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.247
Moisture Content (%)	14.32	14.75
Wet Weight (gms)	138.13	1569.42
Dry Weight (gms)	127.02	1393.42
Container Weight (gms)	49.42	199.94
Density and Saturation		
Wet Weight (gms)	1376.24	
Container Weight (gms)	0.00	
Wet Density (pcf)	134.6	
Dry Density (pcf)	117.7	
Initial Void Ratio	0.431	
% Saturation	89.7	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	98	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	120.0	Initial Burette Ht.(cm)=	49.5
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	19.0
Eff. Consol. Stress (psi) =	90.0	Final Height (in)=	5.937
Induced OCR =	1.0	Initial Volume (cu.in)=	38.951
Change in Ht. of Specimen (in) =	0.0630	Final Volume (cu.in) =	37.090

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	22.64
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	5.77
Failure Criteria:		Eff. Major Principal stress (ksf) =	28.41
Condition at which maximum deviator stress occurs		Axial Strain (%) =	20.21

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	120.0 psi
Project No:	N/A	Back Pressure :	30.0 psi
Boring No.:	MB-1	Consolidation Pressure :	90.0 psi
Depth(ft):	74-75.8	Initial Sample Height:	6.000 in
Sample No.:	SH-8 (B)	Initial Area of Sample:	6.492 sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.937 in
Sample Description:	Sandy Clay	Final Sample Area (A)*:	6.247 sq. in.
		Induced OCR=	1.0

Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
120.0	0	0.000	30.0	0.00	0.00	0.00	0.00	12.96
120.0	253	0.010	32.4	5.82	0.17	0.35	2.91	15.53
120.0	362	0.025	35.3	8.31	0.42	0.76	4.15	16.35
120.0	511	0.050	43.1	11.68	0.84	1.89	5.84	16.91
120.0	561	0.075	50.7	12.77	1.26	2.98	6.38	16.36
120.0	610	0.100	53.7	13.82	1.68	3.41	6.91	16.46
120.0	663	0.150	60.5	14.90	2.53	4.39	7.45	16.02
120.0	703	0.200	66.0	15.66	3.37	5.18	7.83	15.61
120.0	738	0.250	70.2	16.29	4.21	5.79	8.15	15.32
120.0	769	0.300	73.9	16.83	5.05	6.32	8.41	15.05
120.0	798	0.350	76.5	17.31	5.90	6.70	8.65	14.92
120.0	831	0.400	78.6	17.86	6.74	7.00	8.93	14.89
120.0	856	0.450	80.2	18.24	7.58	7.23	9.12	14.85
120.0	888	0.500	81.5	18.74	8.42	7.42	9.37	14.92
120.0	913	0.550	82.6	19.10	9.26	7.57	9.55	14.93
120.0	936	0.600	83.2	19.39	10.11	7.66	9.70	15.00
120.0	965	0.650	83.7	19.81	10.95	7.73	9.90	15.13
120.0	988	0.700	83.9	20.09	11.79	7.76	10.04	15.24
120.0	1016	0.750	83.9	20.46	12.63	7.76	10.23	15.43
120.0	1036	0.800	83.8	20.66	13.47	7.75	10.33	15.54
120.0	1060	0.850	83.6	20.94	14.32	7.72	10.47	15.71
120.0	1085	0.900	83.3	21.22	15.16	7.68	10.61	15.89
120.0	1109	0.950	82.9	21.47	16.00	7.62	10.74	16.08
120.0	1133	1.000	82.4	21.72	16.84	7.55	10.86	16.27
120.0	1152	1.050	81.8	21.86	17.69	7.46	10.93	16.43
120.0	1187	1.100	81.0	22.29	18.53	7.34	11.15	16.76
120.0	1210	1.150	80.4	22.49	19.37	7.26	11.24	16.95
120.0	1231	1.200	79.9	22.64	20.21	7.19	11.32	17.09

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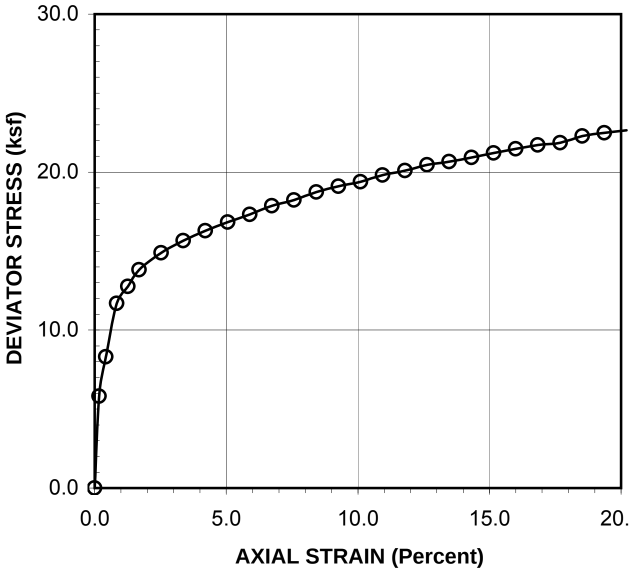
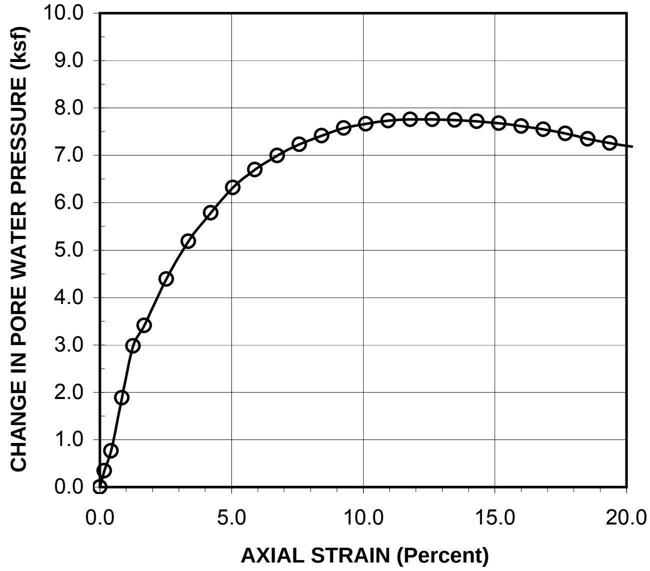
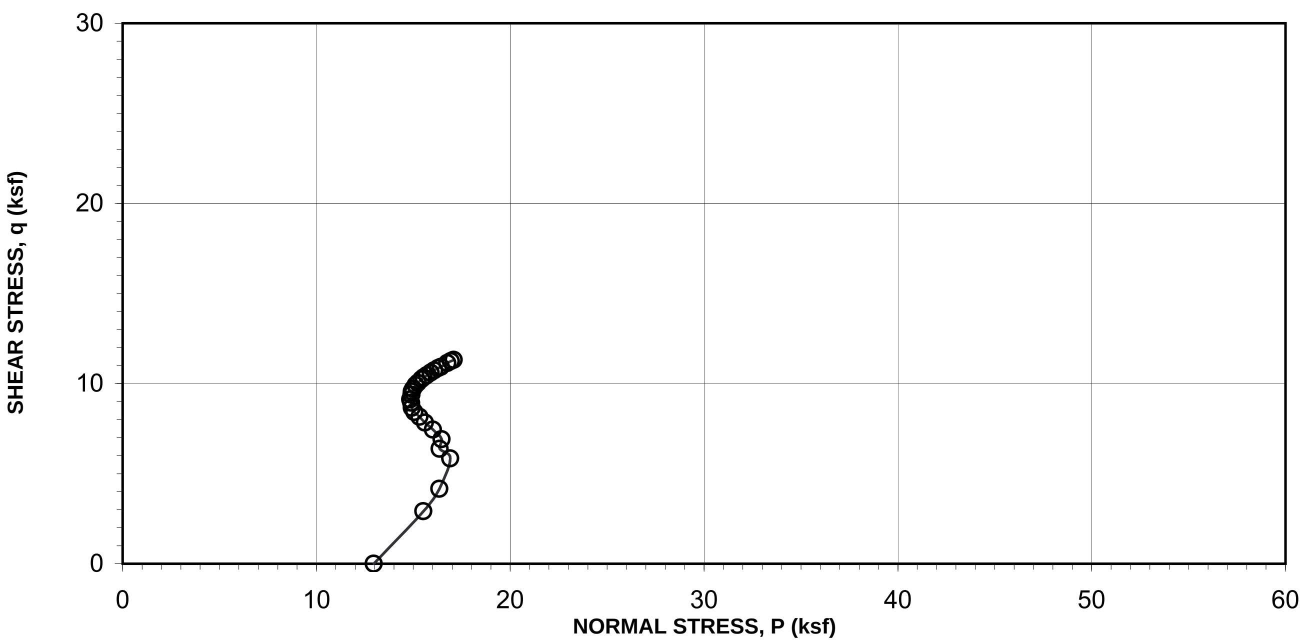




LEGEND: CONFINING PRESSURE= ○ 13 KSF

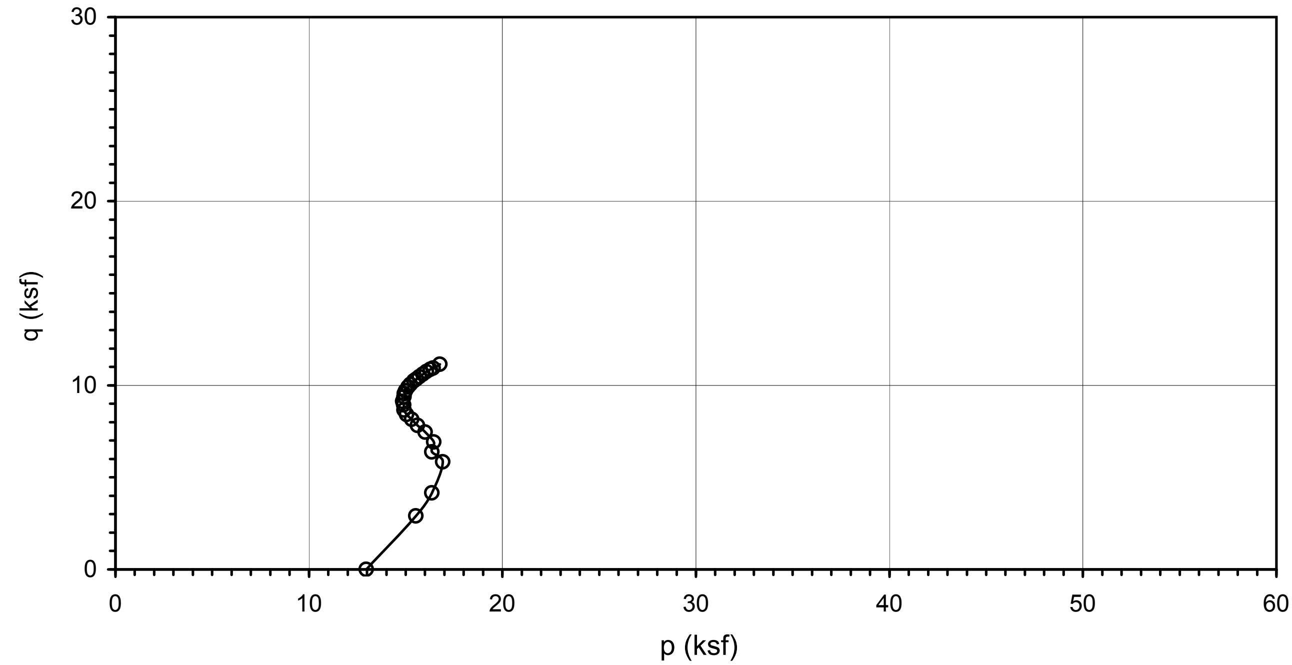


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	117.7
Sample No.:	SH-8 (B)	Initial Moisture Content (%):	14.3
Depth (ft):	74-75.8	Confining Pressure:	13 ksf

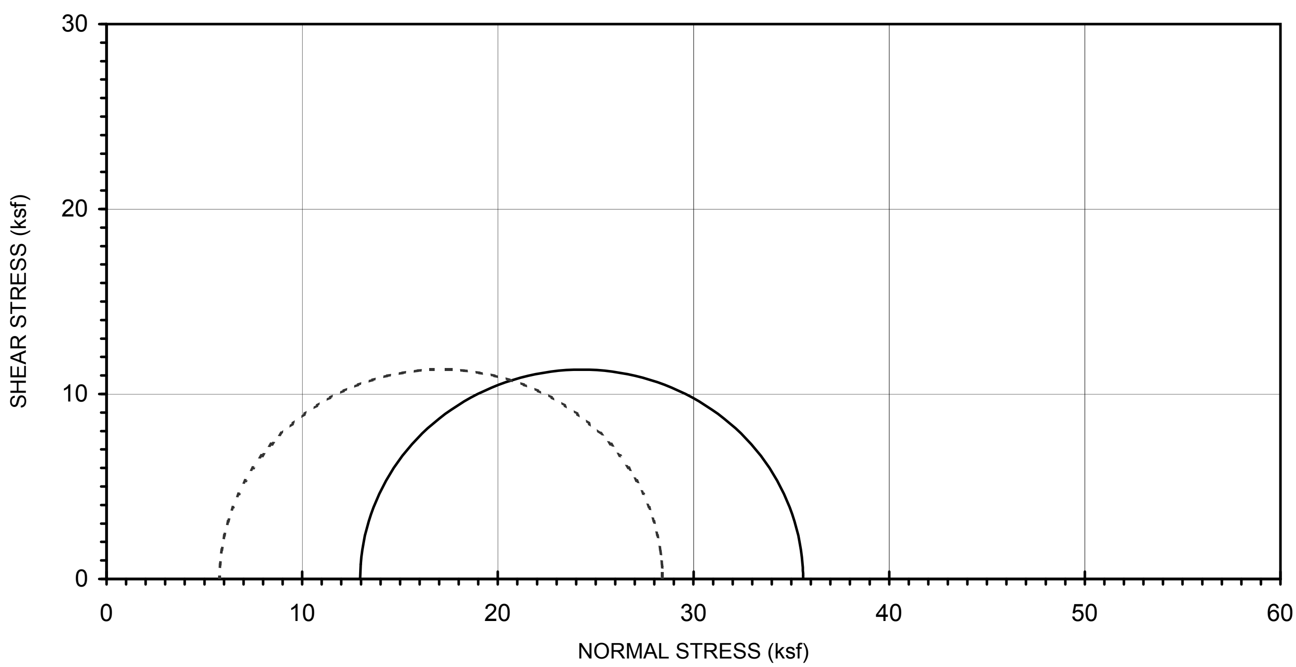
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 12.96 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	117.7
Sample No.:	SH-8 (B)	Initial Moisture Content (%):	14.3
Depth (ft):	74-75.8	Confining Pressure:	13 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT
ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by:	KK	Date:	10-15-08
Project No.:	N/A	Input Data by:	KM	Date:	10-23-08
Boring No.:	MB-1	Reviewed by:	AP	Date:	10-23-08
Sample No.:	SH-8	Sample Description:	Sandy Clay		
Depth(ft):	74-75.8				
Test Condition:	Shelby Tube	Confining Pressure = 8.6 ksf			

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	6.000	6.000	6.000	Avg. =	6.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.383
Moisture Content (%)	14.11	15.39
Wet Weight (gms)	178.00	1533.14
Dry Weight (gms)	162.12	1348.85
Container Weight (gms)	49.61	151.44
Density and Saturation		
Wet Weight (gms)	1379.63	
Container Weight (gms)	0.00	
Wet Density (pcf)	134.9	
Dry Density (pcf)	118.2	
Initial Void Ratio	0.425	
% Saturation	89.7	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	96	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	100.0	Initial Burette Ht.(cm)=	50.1
Back Pressure(psi) =	40.0	Final Burette Ht.(cm)=	31.6
Eff. Consol. Stress (psi) =	60.0	Final Height (in)=	5.925
Induced OCR =	1.0	Initial Volume (cu.in)=	38.951
Change in Ht. of Specimen (in) =	0.0750	Final Volume (cu.in) =	37.822

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	24.45
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	10.05
Failure Criteria:		Eff. Major Principal stress (ksf) =	34.50
Condition at which maximum deviator stress occurs		Axial Strain (%) =	19.41

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	100.0	psi
Project No:	N/A	Back Pressure :	40.0	psi
Boring No.:	MB-1	Consolidation Pressure :	60.0	psi
Depth(ft):	74-75.8	Initial Sample Height:	6.000	in
Sample No.:	SH-8	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.925	in
Sample Description:	Sandy Clay	Final Sample Area (A)*:	6.383	sq. in.
		Induced OCR=	1.0	

Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
100.0	0	0.000	40.0	0.00	0.00	0.00	0.00	8.64
100.0	198	0.010	53.5	4.46	0.17	1.94	2.23	8.93
100.0	273	0.025	59.4	6.13	0.42	2.79	3.07	8.91
100.0	356	0.050	65.9	7.96	0.84	3.73	3.98	8.89
100.0	415	0.075	68.8	9.24	1.27	4.15	4.62	9.11
100.0	475	0.100	70.3	10.53	1.69	4.36	5.27	9.54
100.0	568	0.150	71.7	12.49	2.53	4.56	6.24	10.32
100.0	672	0.200	70.7	14.65	3.38	4.42	7.32	11.54
100.0	760	0.250	68.3	16.42	4.22	4.08	8.21	12.78
100.0	835	0.300	65.5	17.88	5.06	3.67	8.94	13.91
100.0	907	0.350	62.3	19.25	5.91	3.21	9.63	15.05
100.0	962	0.400	59.3	20.24	6.75	2.78	10.12	15.98
100.0	1014	0.450	56.1	21.14	7.59	2.32	10.57	16.89
100.0	1058	0.500	52.8	21.85	8.44	1.84	10.93	17.72
100.0	1099	0.550	49.5	22.49	9.28	1.37	11.25	18.52
100.0	1134	0.600	46.1	22.99	10.13	0.88	11.50	19.26
100.0	1164	0.650	43.2	23.38	10.97	0.46	11.69	19.87
100.0	1191	0.700	40.5	23.69	11.81	0.07	11.85	20.41
100.0	1211	0.750	37.8	23.86	12.66	-0.32	11.93	20.89
100.0	1231	0.800	35.8	24.02	13.50	-0.60	12.01	21.25
100.0	1246	0.850	33.9	24.08	14.35	-0.88	12.04	21.56
100.0	1259	0.900	32.9	24.09	15.19	-1.02	12.04	21.71
100.0	1278	0.950	32.3	24.21	16.03	-1.11	12.10	21.85
100.0	1293	1.000	31.7	24.24	16.88	-1.20	12.12	21.96
100.0	1311	1.050	31.2	24.33	17.72	-1.27	12.17	22.07
100.0	1330	1.100	30.5	24.43	18.57	-1.37	12.22	22.22
100.0	1345	1.150	30.2	24.45	19.41	-1.41	12.23	22.28
100.0	1358	1.200	29.8	24.43	20.25	-1.47	12.21	22.32

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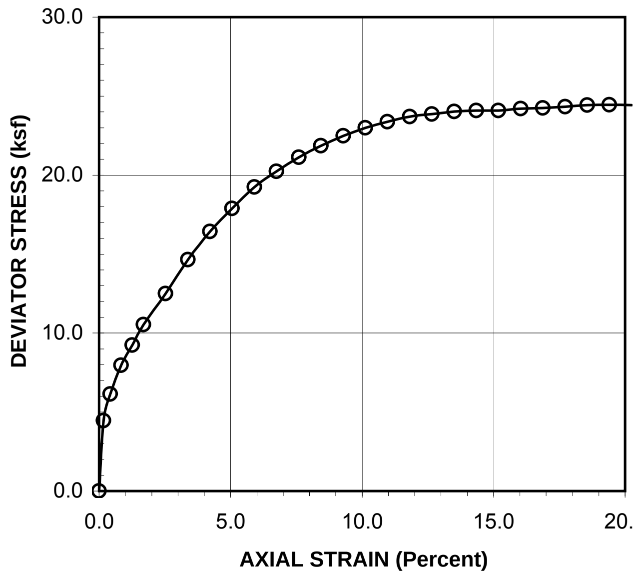
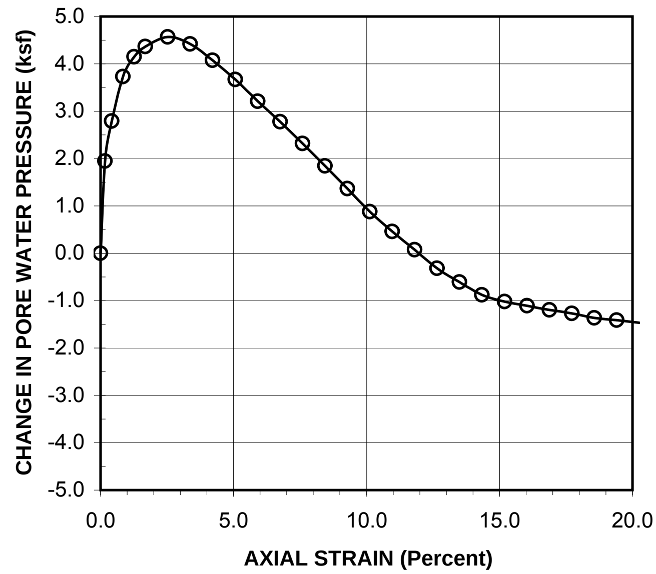
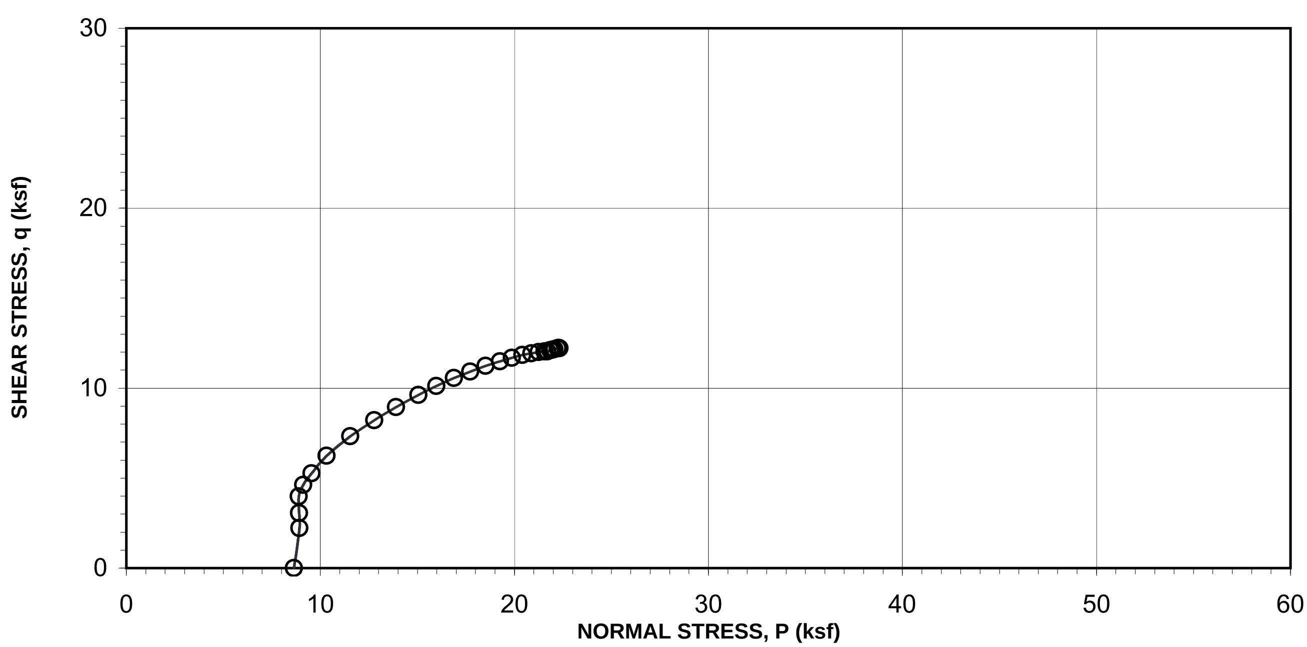




LEGEND: CONFINING PRESSURE= ○ 8.6 KSF

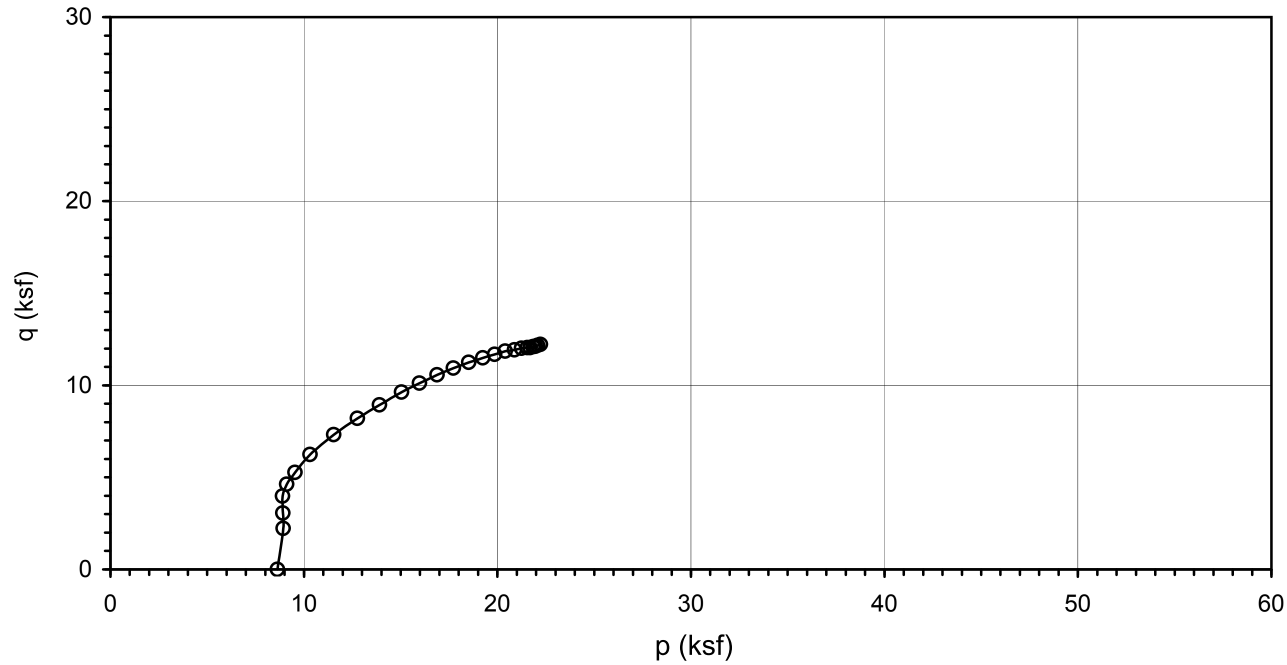


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	118.2
Sample No.:	SH-8	Initial Moisture Content (%):	14.1
Depth (ft):	74-75.8	Confining Pressure:	8.6 ksf

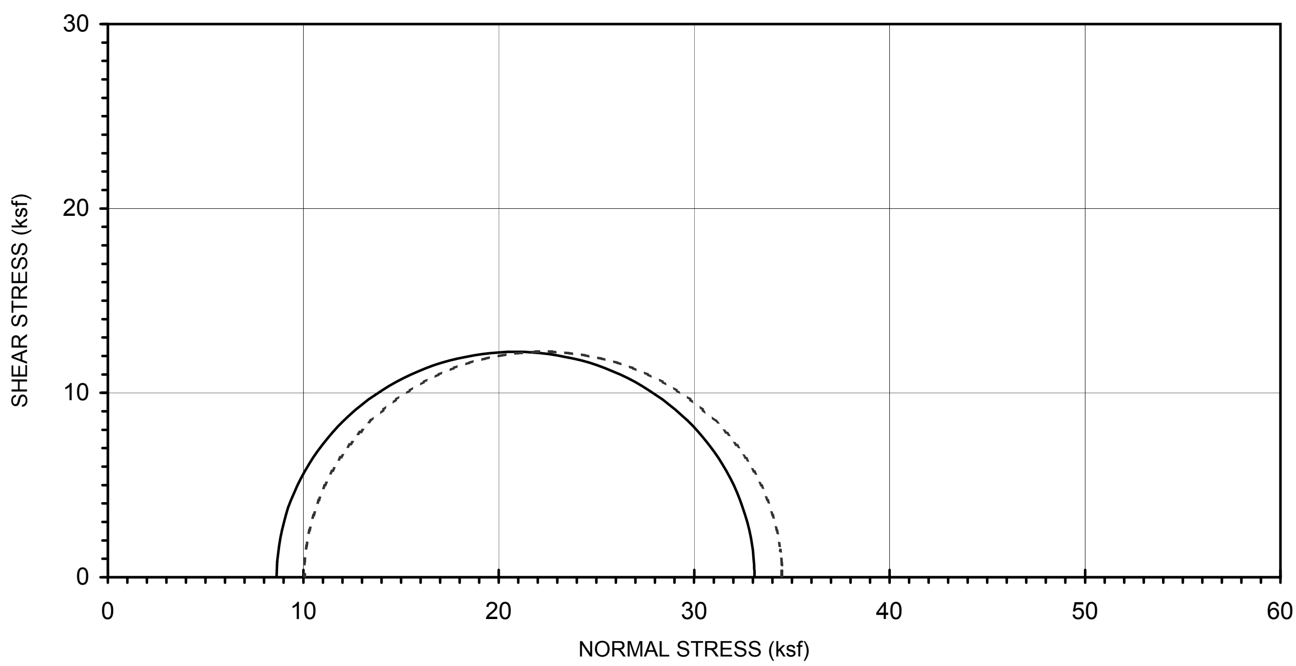
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 8.6 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	118.2
Sample No.:	SH-8	Initial Moisture Content (%):	14.1
Depth (ft):	74-75.8	Confining Pressure:	8.6 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT
ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by:	KK	Date:	10-28-08
Project No.:	N/A	Input Data by:	KM	Date:	11-06-08
Boring No.:	MB-1	Reviewed by:	AP	Date:	11-06-08
Sample No.:	SH-9	Sample Description:	Sandy Clay		
Depth(ft):	78-80				
Test Condition:	Shelby Tube	Confining Pressure = 9.4 ksf			

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	6.000	6.000	6.000	Avg. =	6.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.359
Moisture Content (%)	18.09	18.00
Wet Weight (gms)	194.92	1517.47
Dry Weight (gms)	172.70	1315.34
Container Weight (gms)	49.90	192.64
Density and Saturation		
Wet Weight (gms)	1328.30	
Container Weight (gms)	0.00	
Wet Density (pcf)	129.9	
Dry Density (pcf)	110.0	
Initial Void Ratio	0.532	
% Saturation	91.9	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	98	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	95.0	Initial Burette Ht.(cm)=	46.6
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	21.6
Eff. Consol. Stress (psi) =	65.0	Final Height (in)=	5.885
Induced OCR =	1.0	Initial Volume (cu.in)=	38.951
Change in Ht. of Specimen (in) =	0.1150	Final Volume (cu.in) =	37.426

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	17.05
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	7.82
Failure Criteria:		Eff. Major Principal stress (ksf) =	24.87
Condition at which maximum deviator stress occurs		Axial Strain (%) =	18.69

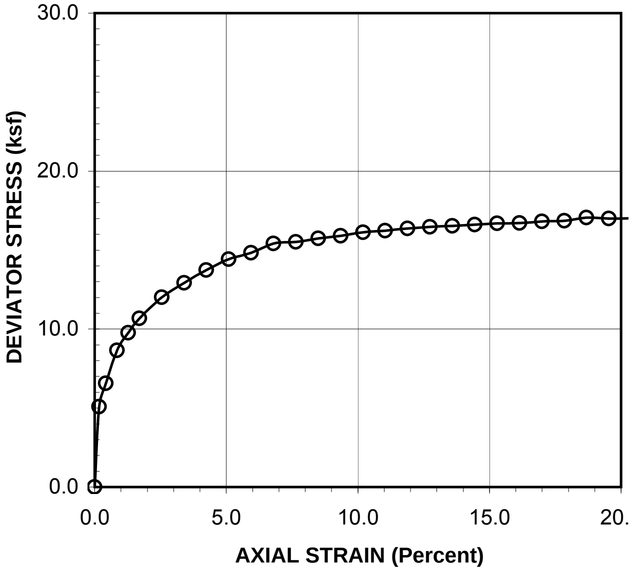
CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

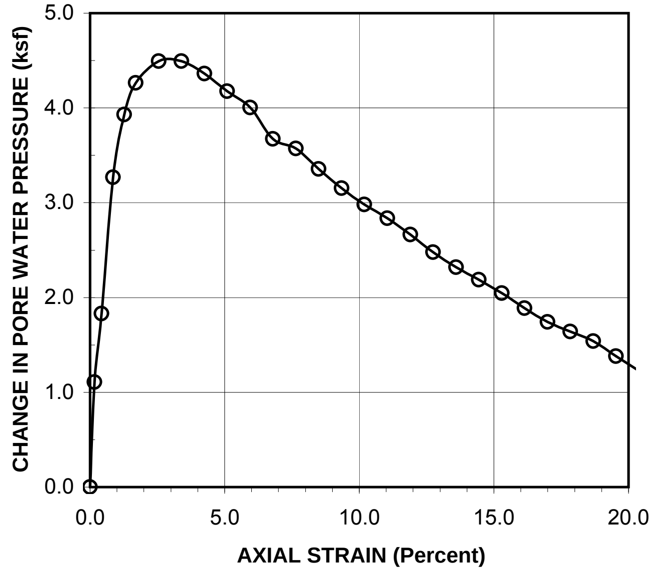
Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	95.0	psi
Project No:	N/A	Back Pressure :	30.0	psi
Boring No.:	MB-1	Consolidation Pressure :	65.0	psi
Depth(ft):	78-80	Initial Sample Height:	6.000	in
Sample No.:	SH-9	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.885	in
Sample Description:	Sandy Clay	Final Sample Area (A)*:	6.359	sq. in.
		Induced OCR=	1.0	

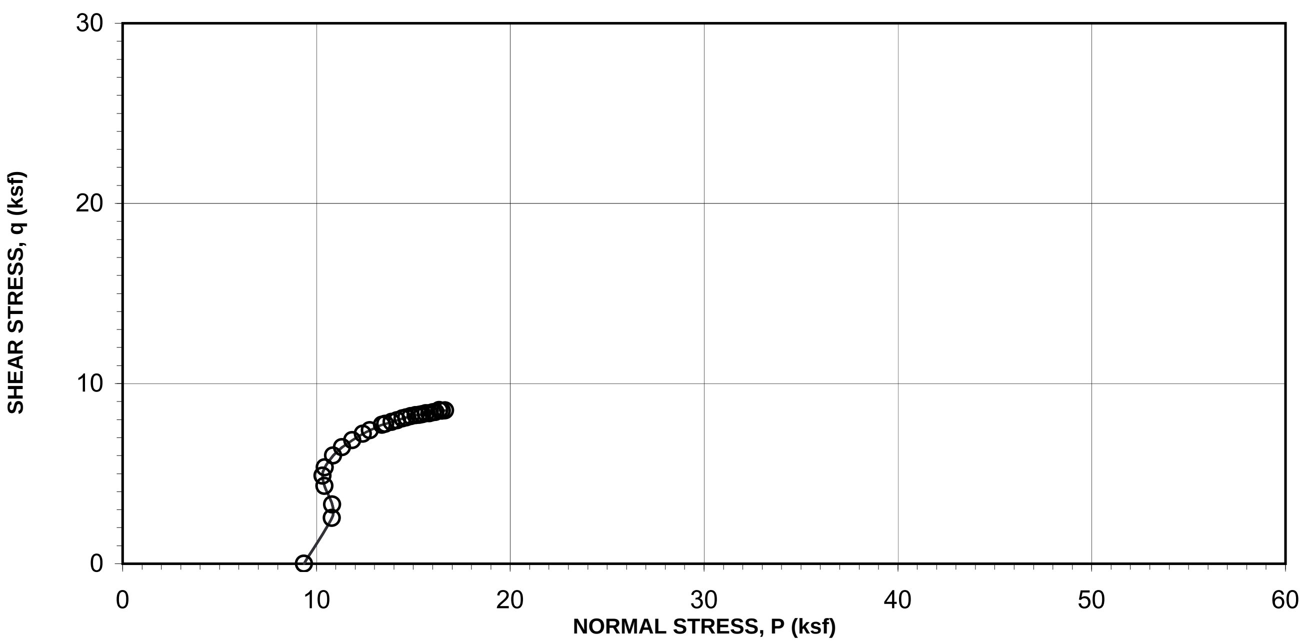
Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
95.0	0	0.000	30.0	0.00	0.00	0.00	0.00	9.36
95.0	225	0.010	37.7	5.09	0.17	1.11	2.54	10.79
95.0	291	0.025	42.7	6.56	0.42	1.83	3.28	10.81
95.0	385	0.050	52.7	8.64	0.85	3.27	4.32	10.41
95.0	437	0.075	57.3	9.77	1.27	3.93	4.88	10.31
95.0	480	0.100	59.6	10.68	1.70	4.26	5.34	10.44
95.0	544	0.150	61.2	12.00	2.55	4.49	6.00	10.87
95.0	591	0.200	61.2	12.93	3.40	4.49	6.46	11.33
95.0	633	0.250	60.3	13.72	4.25	4.36	6.86	11.86
95.0	671	0.300	59.0	14.42	5.10	4.18	7.21	12.39
95.0	696	0.350	57.8	14.82	5.95	4.00	7.41	12.77
95.0	730	0.400	55.5	15.41	6.80	3.67	7.70	13.39
95.0	742	0.450	54.8	15.52	7.65	3.57	7.76	13.55
95.0	760	0.500	53.3	15.75	8.50	3.36	7.87	13.88
95.0	775	0.550	51.9	15.91	9.35	3.15	7.95	14.16
95.0	793	0.600	50.7	16.13	10.20	2.98	8.06	14.44
95.0	806	0.650	49.7	16.23	11.05	2.84	8.12	14.64
95.0	820	0.700	48.5	16.36	11.89	2.66	8.18	14.88
95.0	834	0.750	47.2	16.48	12.74	2.48	8.24	15.12
95.0	845	0.800	46.1	16.53	13.59	2.32	8.27	15.31
95.0	857	0.850	45.2	16.60	14.44	2.19	8.30	15.47
95.0	870	0.900	44.2	16.69	15.29	2.04	8.34	15.66
95.0	880	0.950	43.1	16.71	16.14	1.89	8.35	15.83
95.0	894	1.000	42.1	16.80	16.99	1.74	8.40	16.02
95.0	906	1.050	41.4	16.85	17.84	1.64	8.43	16.15
95.0	926	1.100	40.7	17.05	18.69	1.54	8.52	16.34
95.0	933	1.150	39.6	17.00	19.54	1.38	8.50	16.48
95.0	944	1.200	38.5	17.02	20.39	1.22	8.51	16.64

AP Engineering & Testing, Inc.





LEGEND: CONFINING PRESSURE= ○ 9.4 KSF

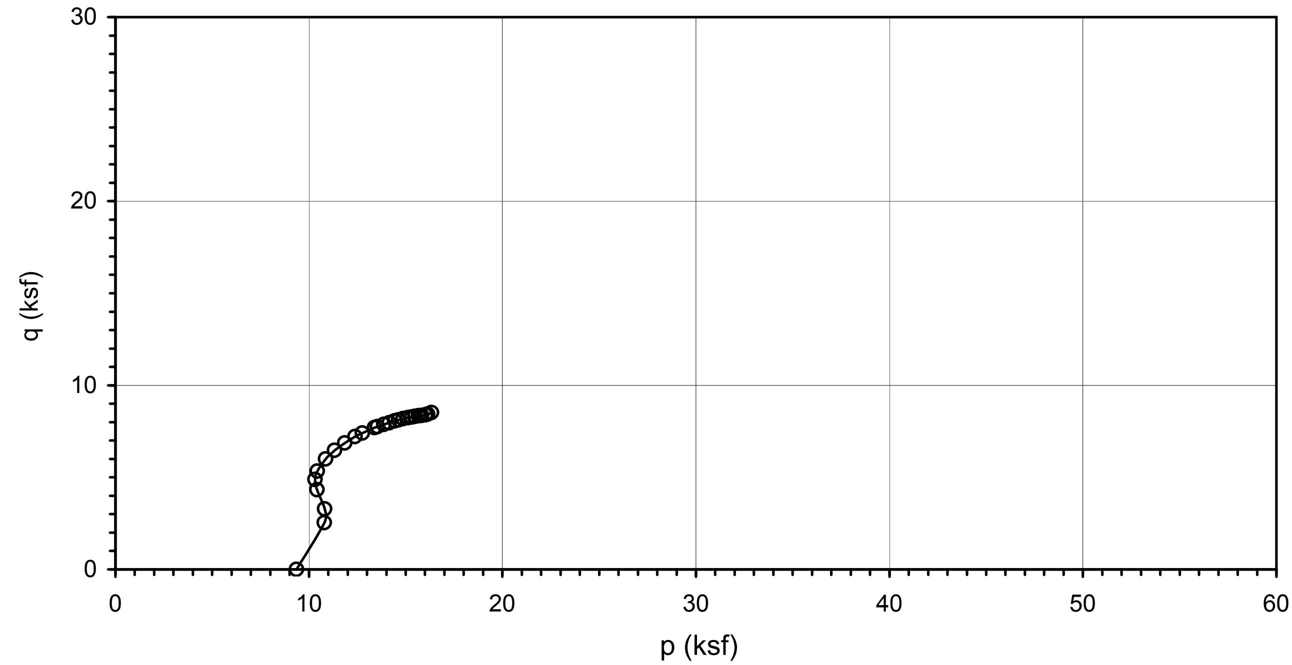


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	110.0
Sample No.:	SH-9	Initial Moisture Content (%):	18.1
Depth (ft):	78-80	Confining Pressure:	9.4 ksf

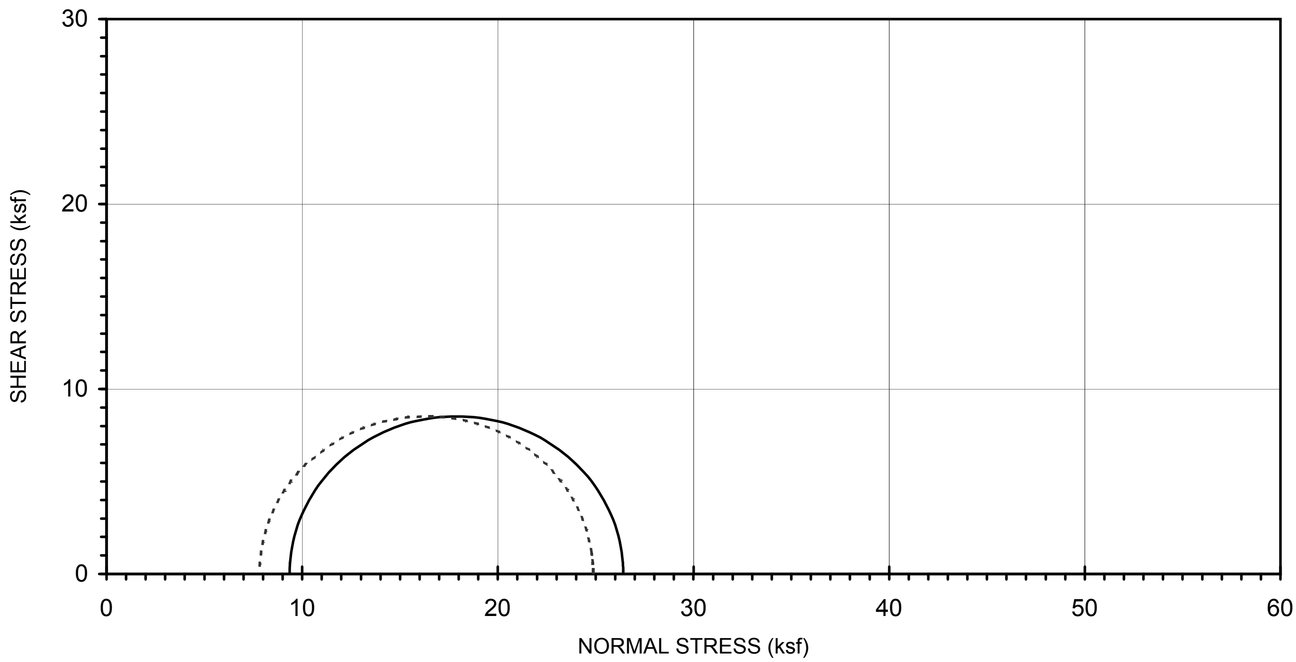
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 8.6 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	110.0
Sample No.:	SH-9	Initial Moisture Content (%):	18.1
Depth (ft):	78-80	Confining Pressure:	9.4 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by: KK	Date: 10-29-08
Project No.:	N/A	Input Data by: KM	Date: 11-06-08
Boring No.:	MB-1	Reviewed by: AP	Date: 11-06-08
Sample No.:	SH-10 (B)	Sample Description: Clayey Sand	
Depth(ft):	88-90		
Test Condition:	Shelby Tube	Confining Pressure = 14.4 ksf	

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	6.000	6.000	6.000	Avg. =	6.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.305
Moisture Content (%)	15.48	12.38
Wet Weight (gms)	202.95	1557.58
Dry Weight (gms)	182.32	1402.96
Container Weight (gms)	49.09	154.09
Density and Saturation		
Wet Weight (gms)	1434.42	
Container Weight (gms)	0.00	
Wet Density (pcf)	140.3	
Dry Density (pcf)	121.5	
Initial Void Ratio	0.387	
% Saturation	108.1	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	98	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	130.0	Initial Burette Ht.(cm)=	47.1
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	15.3
Eff. Consol. Stress (psi) =	100.0	Final Height (in)=	5.870
Induced OCR =	1.0	Initial Volume (cu.in)=	38.951
Change in Ht. of Specimen (in) =	0.1300	Final Volume (cu.in) =	37.011

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	23.48
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	10.41
Failure Criteria:		Eff. Major Principal stress (ksf) =	33.89
Condition at which maximum deviator stress occurs		Axial Strain (%) =	20.44

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	130.0	psi
Project No:	N/A	Back Pressure :	30.0	psi
Boring No.:	MB-1	Consolidation Pressure :	100.0	psi
Depth(ft):	88-90	Initial Sample Height:	6.000	in
Sample No.:	SH-10 (B)	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.870	in
Sample Description:	Clayey Sand	Final Sample Area (A)*:	6.305	sq. in.
		Induced OCR=	1.0	

Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
130.0	0	0.000	30.0	0.00	0.00	0.00	0.00	14.40
130.0	210	0.010	47.3	4.79	0.17	2.49	2.39	14.30
130.0	266	0.025	58.5	6.05	0.43	4.10	3.02	13.32
130.0	332	0.050	74.1	7.51	0.85	6.35	3.76	11.81
130.0	367	0.075	87.4	8.27	1.28	8.27	4.14	10.27
130.0	396	0.100	93.2	8.89	1.70	9.10	4.45	9.75
130.0	450	0.150	99.6	10.02	2.56	10.02	5.01	9.39
130.0	503	0.200	102.5	11.09	3.41	10.44	5.54	9.50
130.0	554	0.250	103.2	12.12	4.26	10.54	6.06	9.92
130.0	605	0.300	102.7	13.11	5.11	10.47	6.55	10.48
130.0	656	0.350	101.1	14.09	5.96	10.24	7.04	11.21
130.0	697	0.400	99.1	14.82	6.81	9.95	7.41	11.86
130.0	751	0.450	97.1	15.84	7.67	9.66	7.92	12.66
130.0	797	0.500	94.6	16.66	8.52	9.30	8.33	13.43
130.0	839	0.550	92.0	17.36	9.37	8.93	8.68	14.15
130.0	889	0.600	89.0	18.23	10.22	8.50	9.11	15.02
130.0	928	0.650	86.7	18.84	11.07	8.16	9.42	15.65
130.0	975	0.700	83.4	19.61	11.93	7.69	9.81	16.52
130.0	1007	0.750	81.1	20.05	12.78	7.36	10.03	17.07
130.0	1045	0.800	78.5	20.62	13.63	6.98	10.31	17.72
130.0	1083	0.850	75.8	21.15	14.48	6.60	10.58	18.38
130.0	1115	0.900	73.1	21.56	15.33	6.21	10.78	18.98
130.0	1152	0.950	70.0	22.06	16.18	5.76	11.03	19.67
130.0	1178	1.000	67.3	22.32	17.04	5.37	11.16	20.19
130.0	1208	1.050	65.2	22.66	17.89	5.07	11.33	20.66
130.0	1238	1.100	62.9	22.98	18.74	4.74	11.49	21.15
130.0	1265	1.150	60.3	23.23	19.59	4.36	11.61	21.65
130.0	1292	1.200	57.7	23.48	20.44	3.99	11.74	22.15

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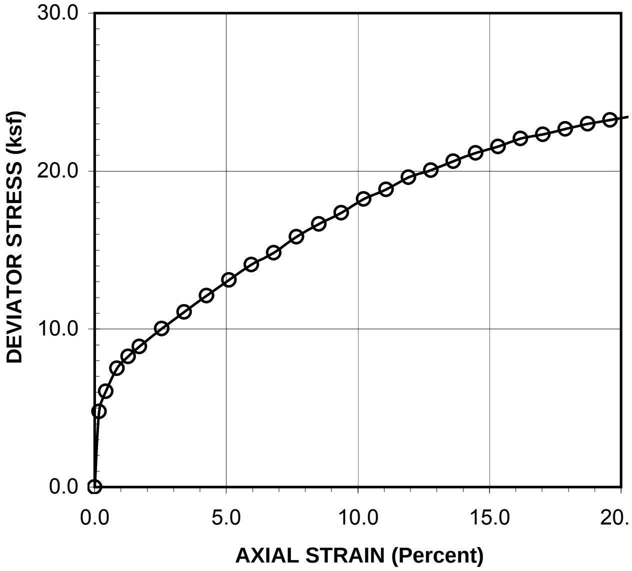
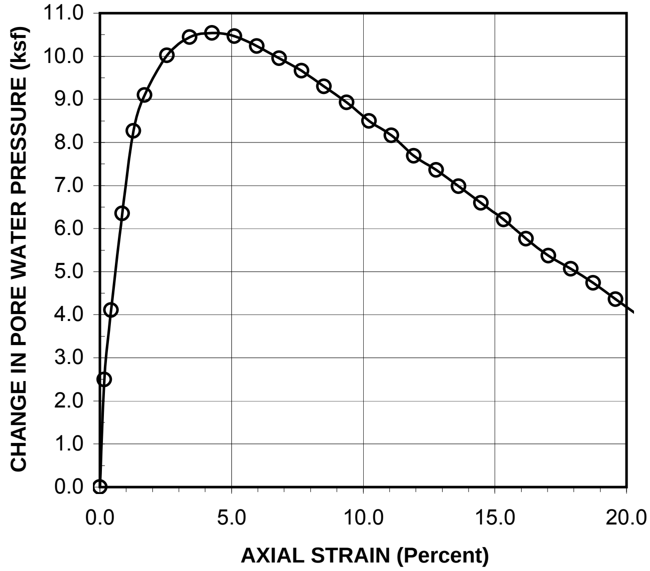
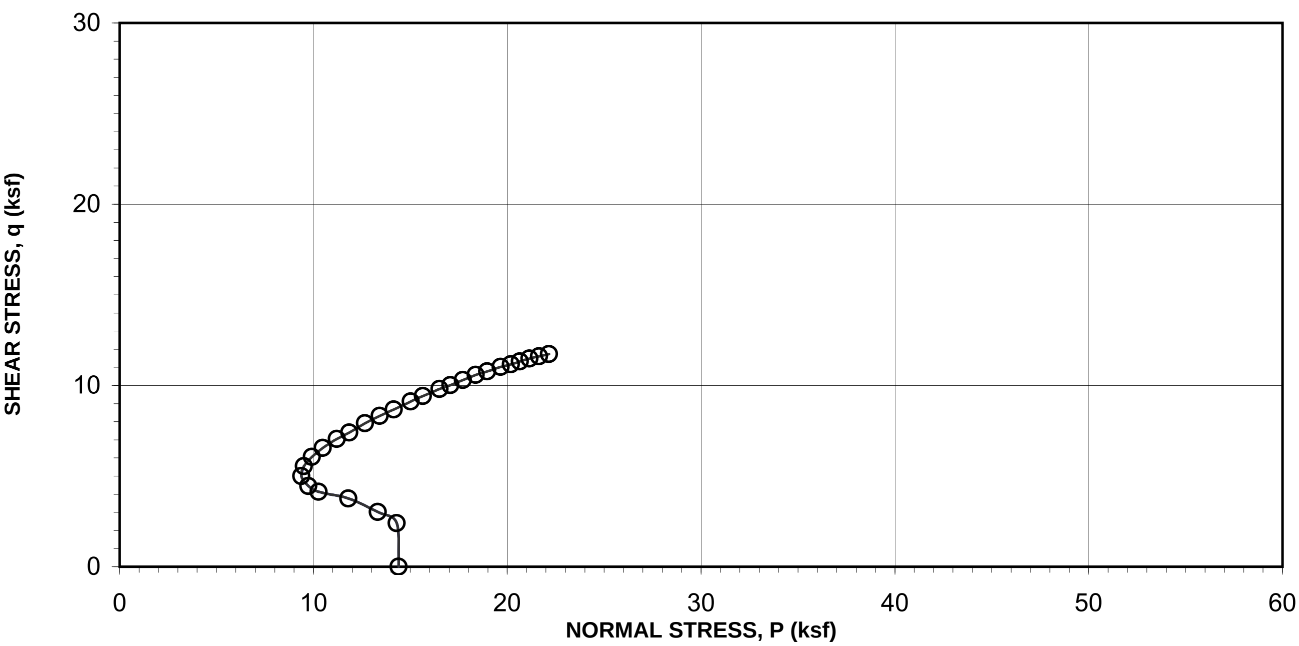




LEGEND: CONFINING PRESSURE= ○ 14.4 KSF

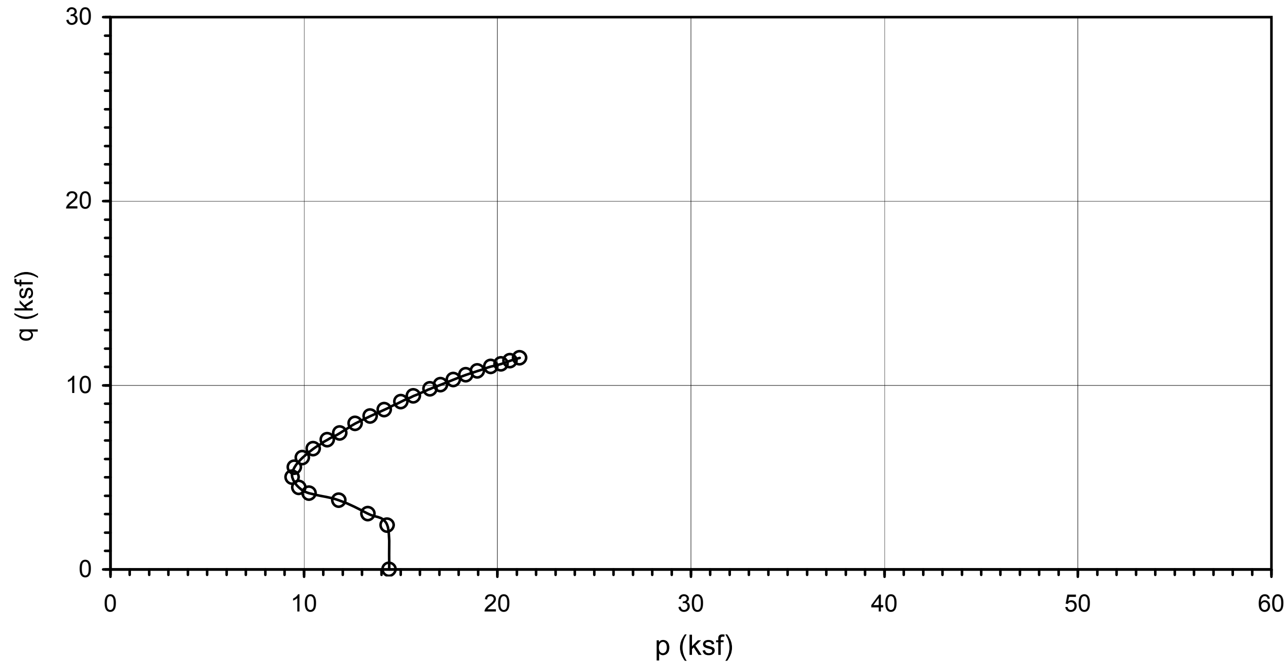


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Clayey Sand
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	121.5
Sample No.:	SH-10 (B)	Initial Moisture Content (%):	15.5
Depth (ft):	88-90	Confining Pressure:	14.4 ksf

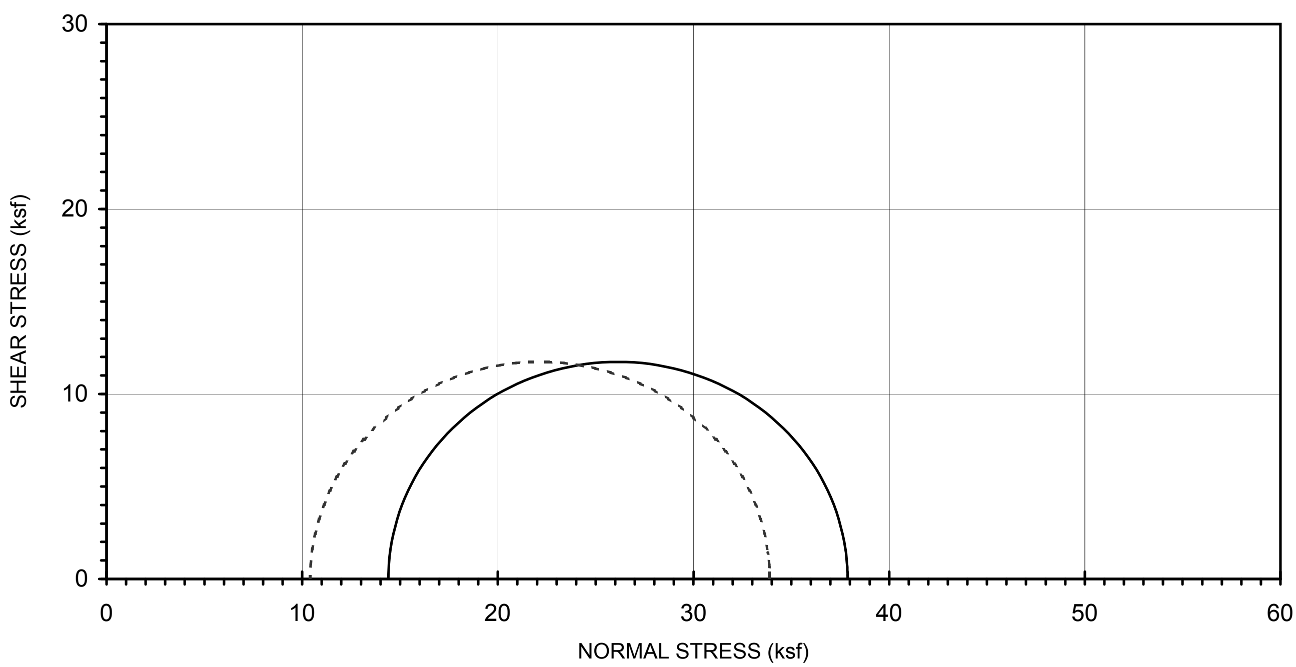
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 14.4 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Clayey Sand
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	121.5
Sample No.:	SH-10 (B)	Initial Moisture Content (%):	15.5
Depth (ft):	88-90	Confining Pressure:	14.4 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by: KK	Date: 10-15-08
Project No.:	N/A	Input Data by: KM	Date: 10-23-08
Boring No.:	MB-1	Reviewed by: AP	Date: 10-23-08
Sample No.:	SH-10	Sample Description: Sandy Clay	
Depth(ft):	88-90		
Test Condition:	Shelby Tube	Confining Pressure = 10.0 ksf	

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	5.750	5.750	5.750	Avg. =	5.750

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.406
Moisture Content (%)	14.95	13.81
Wet Weight (gms)	159.34	1541.47
Dry Weight (gms)	144.91	1378.24
Container Weight (gms)	48.41	196.56
Density and Saturation		
Wet Weight (gms)	1364.23	
Container Weight (gms)	0.00	
Wet Density (pcf)	139.2	
Dry Density (pcf)	121.1	
Initial Void Ratio	0.391	
% Saturation	103.2	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	96	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	110.0	Initial Burette Ht.(cm)=	21.0
Back Pressure(psi) =	40.0	Final Burette Ht.(cm)=	4.0
Eff. Consol. Stress (psi) =	70.0	Final Height (in)=	5.665
Induced OCR =	1.0	Initial Volume (cu.in)=	37.328
Change in Ht. of Specimen (in) =	0.0850	Final Volume (cu.in) =	36.291

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	18.11
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	7.91
Failure Criteria:		Eff. Major Principal stress (ksf) =	26.02
Condition at which maximum deviator stress occurs		Axial Strain (%) =	20.30

AP Engineering & Testing, Inc.

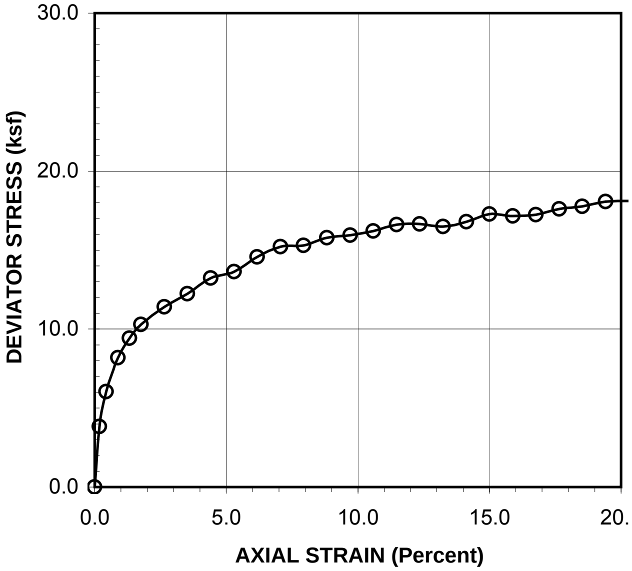
CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

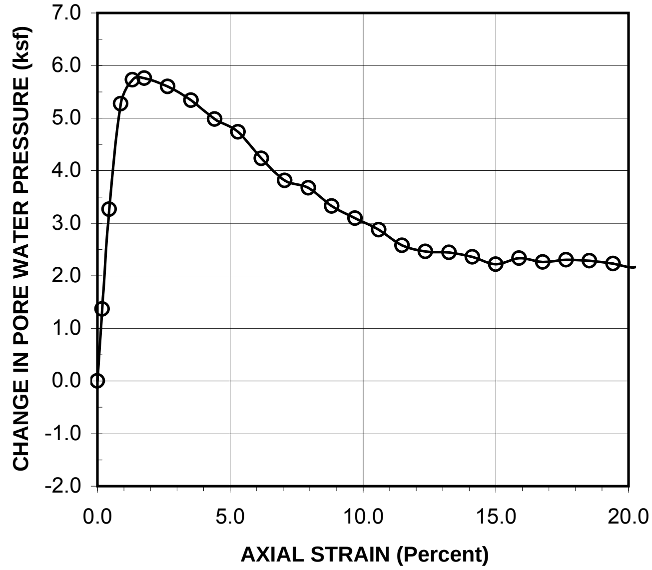
Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	110.0	psi
Project No:	N/A	Back Pressure :	40.0	psi
Boring No.:	MB-1	Consolidation Pressure :	70.0	psi
Depth(ft):	88-90	Initial Sample Height:	5.750	in
Sample No.:	SH-10	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.665	in
Sample Description:	Sandy Clay	Final Sample Area (A)*:	6.406	sq. in.
		Induced OCR=	1.0	

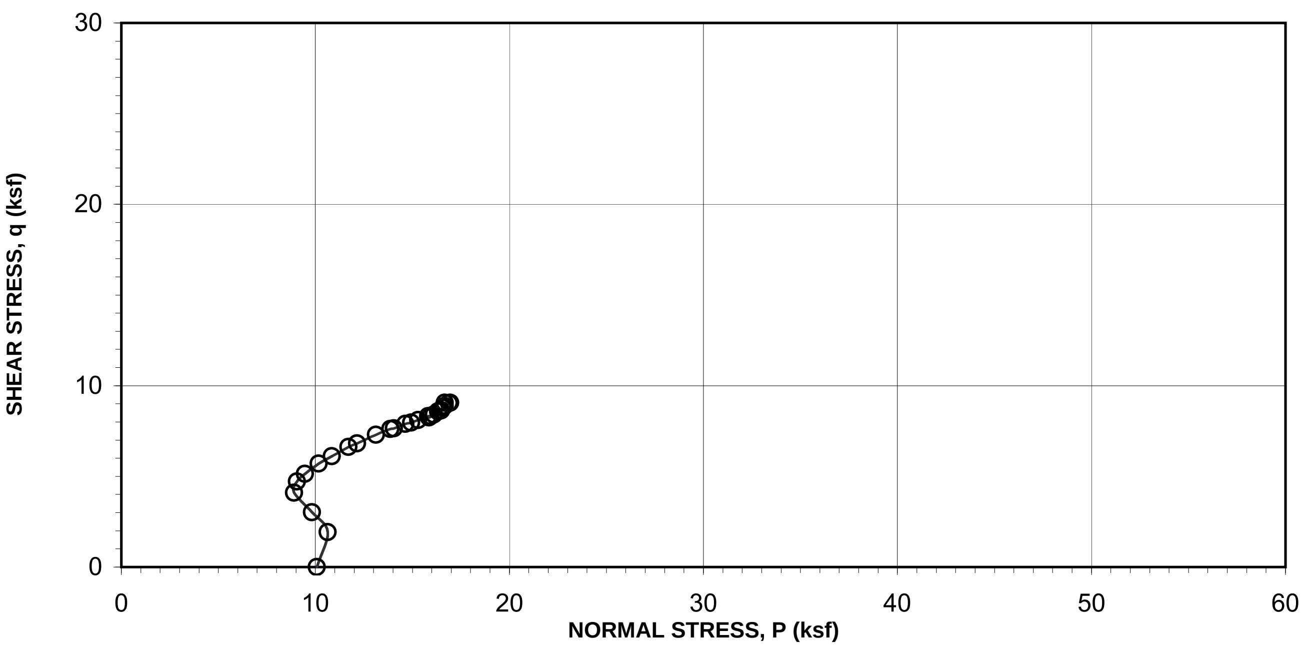
Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
110.0	0	0.000	40.0	0.00	0.00	0.00	0.00	10.08
110.0	171	0.010	49.5	3.84	0.18	1.37	1.92	10.63
110.0	270	0.025	62.7	6.04	0.44	3.27	3.02	9.83
110.0	367	0.050	76.6	8.18	0.88	5.27	4.09	8.90
110.0	425	0.075	79.8	9.43	1.32	5.73	4.71	9.06
110.0	466	0.100	80.0	10.29	1.77	5.76	5.15	9.47
110.0	521	0.150	78.9	11.40	2.65	5.60	5.70	10.18
110.0	564	0.200	77.1	12.23	3.53	5.34	6.12	10.85
110.0	616	0.250	74.6	13.24	4.41	4.98	6.62	11.72
110.0	640	0.300	72.9	13.62	5.30	4.74	6.81	12.15
110.0	691	0.350	69.4	14.57	6.18	4.23	7.29	13.13
110.0	728	0.400	66.5	15.21	7.06	3.82	7.60	13.87
110.0	739	0.450	65.5	15.29	7.94	3.67	7.65	14.05
110.0	770	0.500	63.1	15.78	8.83	3.33	7.89	14.64
110.0	785	0.550	61.5	15.93	9.71	3.10	7.97	14.95
110.0	806	0.600	60.0	16.20	10.59	2.88	8.10	15.30
110.0	835	0.650	57.9	16.62	11.47	2.58	8.31	15.81
110.0	845	0.700	57.1	16.65	12.36	2.46	8.32	15.94
110.0	845	0.750	57.0	16.48	13.24	2.45	8.24	15.87
110.0	870	0.800	56.4	16.80	14.12	2.36	8.40	16.12
110.0	904	0.850	55.4	17.27	15.00	2.22	8.64	16.50
110.0	907	0.900	56.2	17.15	15.89	2.33	8.57	16.32
110.0	921	0.950	55.7	17.23	16.77	2.26	8.62	16.43
110.0	951	1.000	56.0	17.60	17.65	2.30	8.80	16.58
110.0	970	1.050	55.9	17.76	18.53	2.29	8.88	16.67
110.0	998	1.100	55.5	18.08	19.42	2.23	9.04	16.89
110.0	1011	1.150	55.1	18.11	20.30	2.17	9.06	16.96
110.0	1022	1.200	57.1	18.11	21.18	2.46	9.05	16.67

AP Engineering & Testing, Inc.





LEGEND: CONFINING PRESSURE= ○ 10 KSF

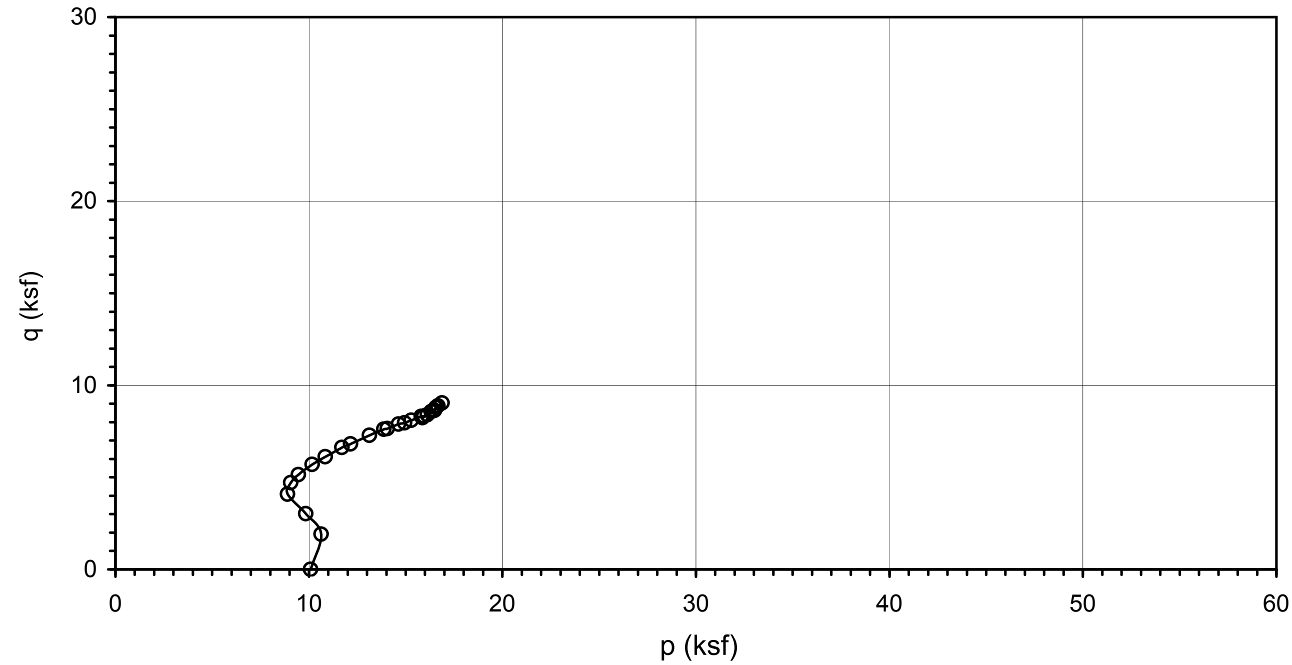


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	121.1
Sample No.:	SH-10	Initial Moisture Content (%):	15.0
Depth (ft):	88-90	Confining Pressure:	10 ksf

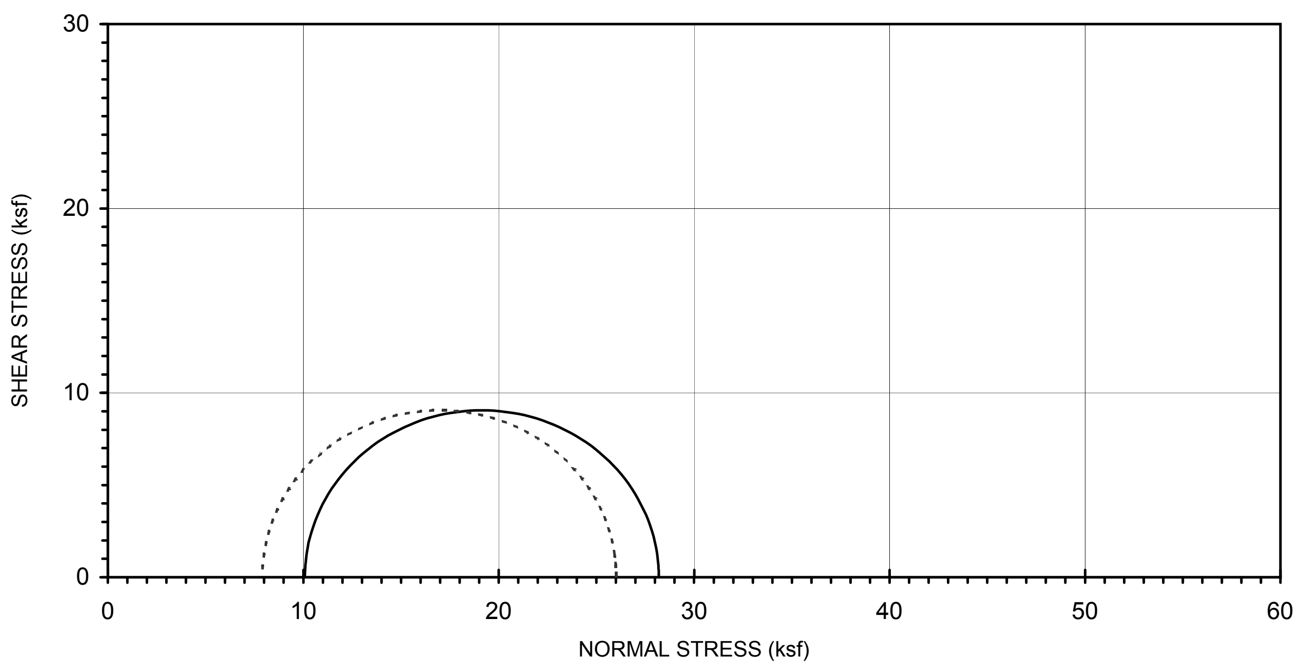
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 10 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	121.1
Sample No.:	SH-10	Initial Moisture Content (%):	15.0
Depth (ft):	88-90	Confining Pressure:	10 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by: KK	Date: 10-21-08
Project No.:	N/A	Input Data by: KM	Date: 10-23-08
Boring No.:	MB-1	Reviewed by: AP	Date: 10-23-08
Sample No.:	SH-11 (B)	Sample Description: Sandy Clay	
Depth(ft):	98-99.5		
Test Condition:	Shelby Tube	Confining Pressure = 15.8 ksf	

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	6.000	6.000	6.000	Avg. =	6.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.303
Moisture Content (%)	11.59	13.33
Wet Weight (gms)	161.11	1593.92
Dry Weight (gms)	149.56	1429.49
Container Weight (gms)	49.89	195.80
Density and Saturation		
Wet Weight (gms)	1405.59	
Container Weight (gms)	0.00	
Wet Density (pcf)	137.5	
Dry Density (pcf)	123.2	
Initial Void Ratio	0.368	
% Saturation	85.1	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	94	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	140.0	Initial Burette Ht.(cm)=	65.2
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	38.7
Eff. Consol. Stress (psi) =	110.0	Final Height (in)=	5.923
Induced OCR =	1.0	Initial Volume (cu.in)=	38.951
Change in Ht. of Specimen (in) =	0.0770	Final Volume (cu.in) =	37.334

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	31.82
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	13.45
Failure Criteria:		Eff. Major Principal stress (ksf) =	45.27
Condition at which maximum deviator stress occurs		Axial Strain (%) =	19.42

AP Engineering & Testing, Inc.

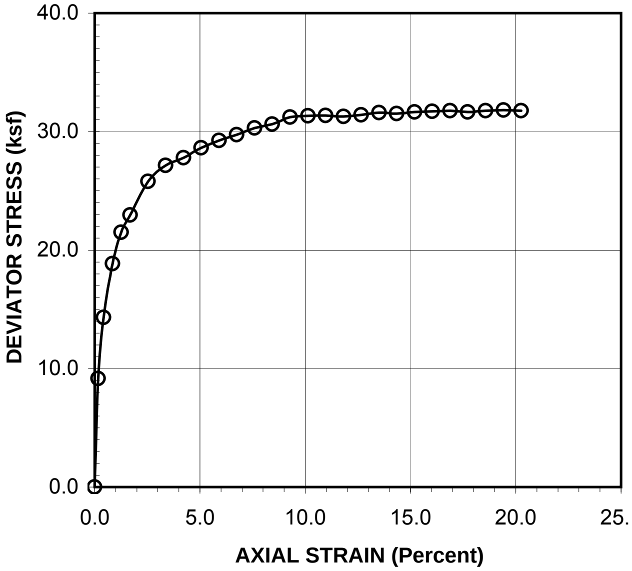
CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

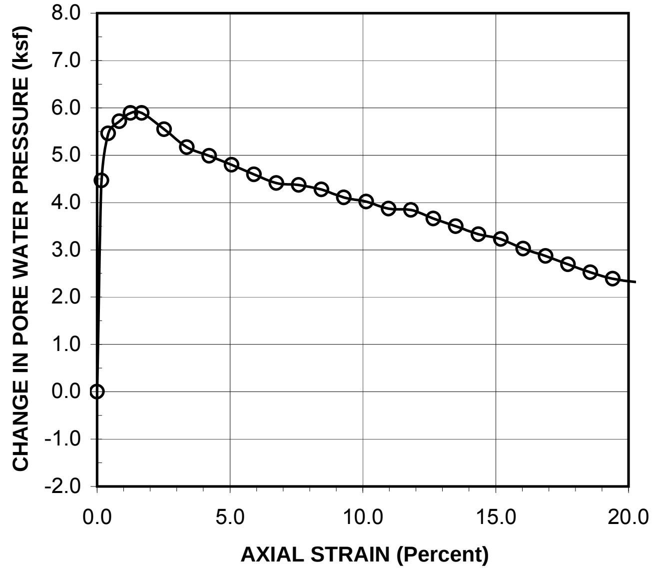
Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	140.0	psi
Project No:	N/A	Back Pressure :	30.0	psi
Boring No.:	MB-1	Consolidation Pressure :	110.0	psi
Depth(ft):	98-99.5	Initial Sample Height:	6.000	in
Sample No.:	SH-11 (B)	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.923	in
Sample Description:	Sandy Clay	Final Sample Area (A)*:	6.303	sq. in.
		Induced OCR=	1.0	

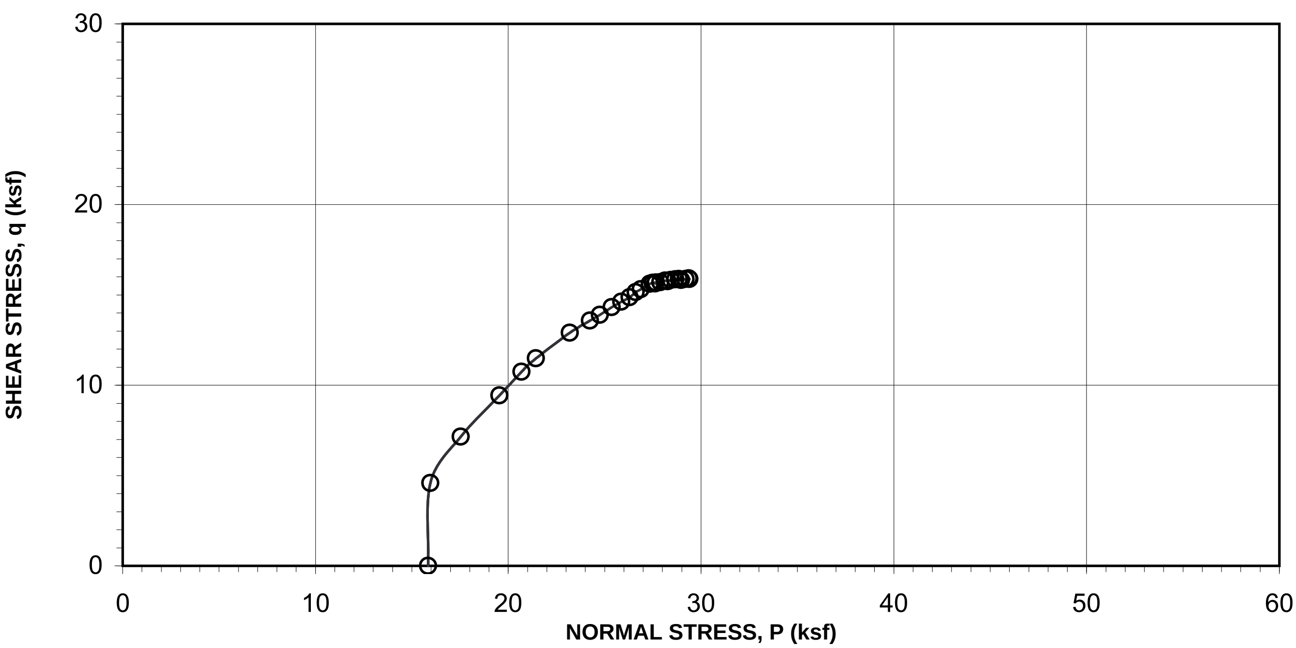
Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
140.0	0	0.000	0.0	0.00	0.00	0.00	0.00	15.84
140.0	402	0.010	31.0	9.16	0.17	4.46	4.58	15.96
140.0	629	0.025	37.9	14.31	0.42	5.46	7.15	17.53
140.0	833	0.050	39.7	18.86	0.84	5.72	9.43	19.55
140.0	953	0.075	40.9	21.49	1.27	5.89	10.74	20.69
140.0	1022	0.100	40.9	22.96	1.69	5.89	11.48	21.43
140.0	1158	0.150	38.5	25.79	2.53	5.54	12.90	23.19
140.0	1230	0.200	35.9	27.14	3.38	5.17	13.57	24.24
140.0	1270	0.250	34.6	27.79	4.22	4.98	13.90	24.75
140.0	1320	0.300	33.3	28.64	5.07	4.80	14.32	25.36
140.0	1361	0.350	31.9	29.25	5.91	4.59	14.63	25.87
140.0	1396	0.400	30.6	29.75	6.75	4.41	14.87	26.31
140.0	1435	0.450	30.3	30.30	7.60	4.37	15.15	26.62
140.0	1464	0.500	29.7	30.63	8.44	4.28	15.32	26.88
140.0	1507	0.550	28.5	31.22	9.29	4.10	15.61	27.35
140.0	1526	0.600	27.9	31.33	10.13	4.02	15.67	27.49
140.0	1542	0.650	26.9	31.37	10.97	3.87	15.68	27.65
140.0	1552	0.700	26.7	31.27	11.82	3.84	15.63	27.63
140.0	1574	0.750	25.4	31.41	12.66	3.66	15.70	27.89
140.0	1599	0.800	24.3	31.59	13.51	3.50	15.79	28.14
140.0	1611	0.850	23.1	31.52	14.35	3.33	15.76	28.27
140.0	1634	0.900	22.4	31.65	15.20	3.23	15.83	28.44
140.0	1653	0.950	21.0	31.70	16.04	3.02	15.85	28.67
140.0	1673	1.000	19.9	31.77	16.88	2.87	15.88	28.86
140.0	1684	1.050	18.7	31.64	17.73	2.69	15.82	28.97
140.0	1707	1.100	17.5	31.76	18.57	2.52	15.88	29.20
140.0	1729	1.150	16.6	31.82	19.42	2.39	15.91	29.36
140.0	1743	1.200	16.1	31.75	20.26	2.32	15.88	29.40

AP Engineering & Testing, Inc.





LEGEND: CONFINING PRESSURE= ○ 15.8 KSF

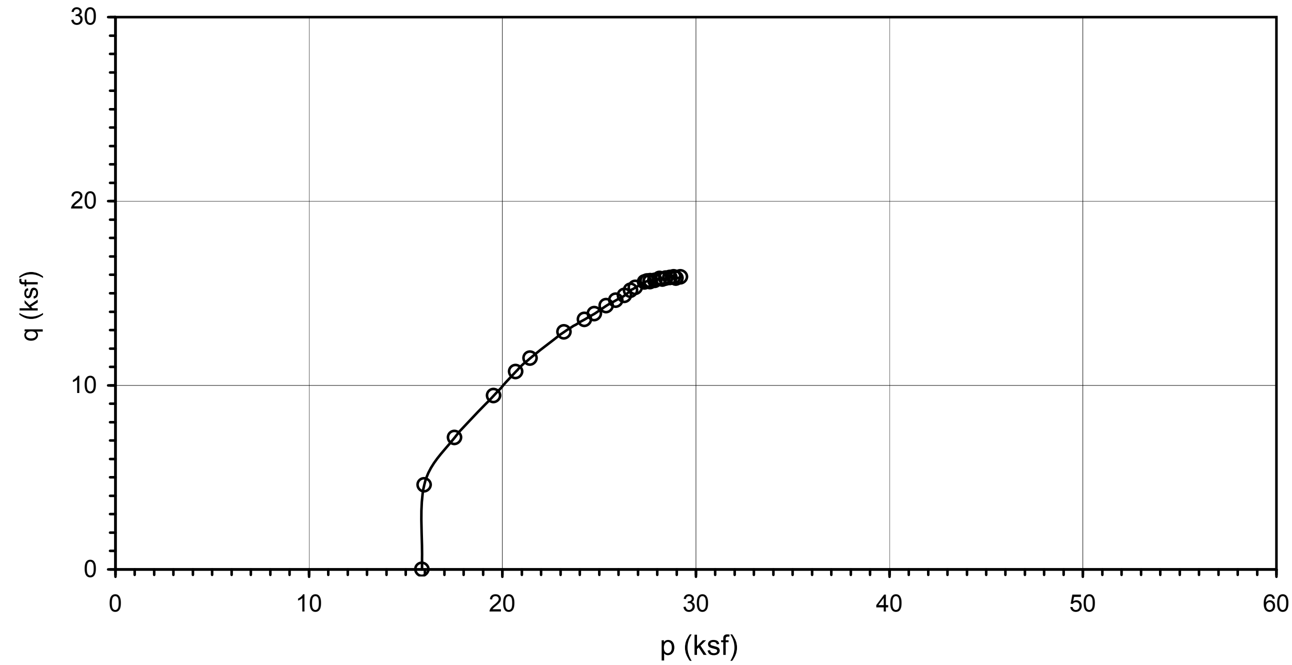


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	123.2
Sample No.:	SH-11 (B)	Initial Moisture Content (%):	11.6
Depth (ft):	98-99.5	Confining Pressure:	15.8 ksf

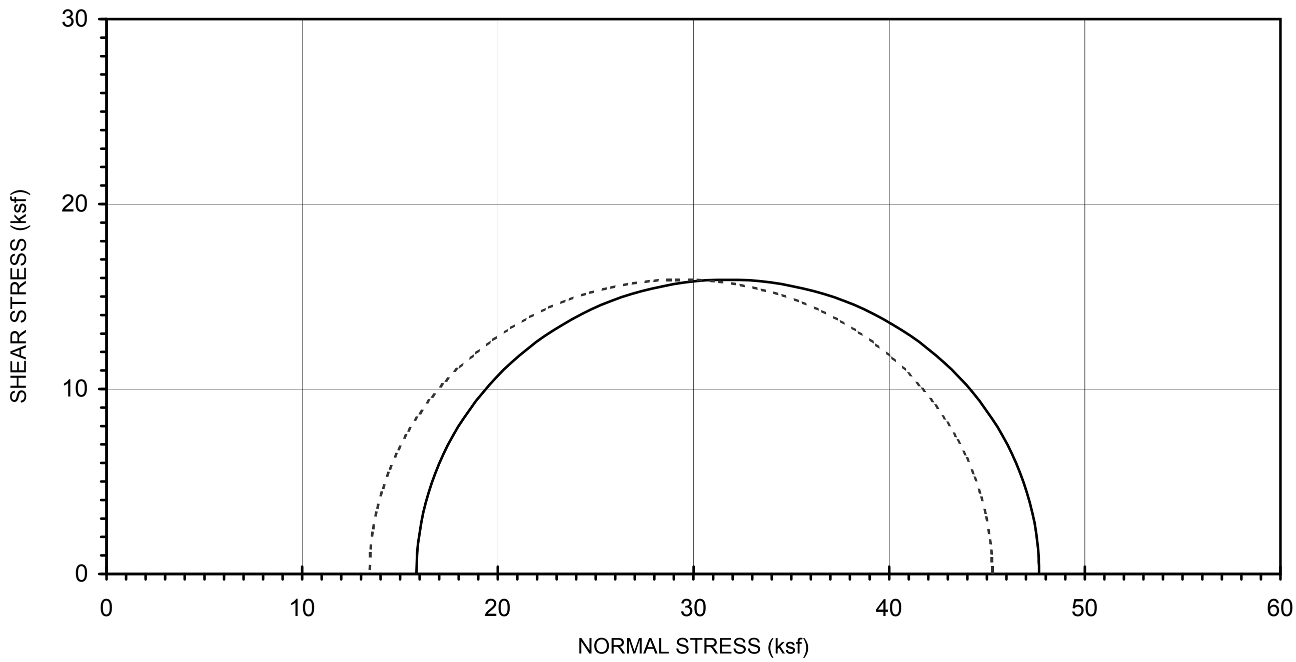
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 15.8 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	123.2
Sample No.:	SH-11 (B)	Initial Moisture Content (%):	11.6
Depth (ft):	98-99.5	Confining Pressure:	15.8 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT
ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by: KK	Date: 10-21-08
Project No.:	N/A	Input Data by: KM	Date: 10-23-08
Boring No.:	MB-1	Reviewed by: AP	Date: 10-23-08
Sample No.:	SH-11	Sample Description: Sandy Clay	
Depth(ft):	98-99.5		
Test Condition:	Shelby Tube	Confining Pressure = 10.8 ksf	

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	6.000	6.000	6.000	Avg. =	6.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.251
Moisture Content (%)	10.00	15.13
Wet Weight (gms)	150.46	1577.81
Dry Weight (gms)	141.32	1395.83
Container Weight (gms)	49.95	193.35
Density and Saturation		
Wet Weight (gms)	1390.04	
Container Weight (gms)	0.00	
Wet Density (pcf)	136.0	
Dry Density (pcf)	123.6	
Initial Void Ratio	0.363	
% Saturation	74.4	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	95	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	105.0	Initial Burette Ht.(cm)=	54.0
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	27.1
Eff. Consol. Stress (psi) =	75.0	Final Height (in)=	5.969
Induced OCR =	1.0	Initial Volume (cu.in)=	38.951
Change in Ht. of Specimen (in) =	0.0310	Final Volume (cu.in) =	37.310

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	23.83
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	8.73
Failure Criteria:		Eff. Major Principal stress (ksf) =	32.56
Condition at which maximum deviator stress occurs		Axial Strain (%) =	18.43

AP Engineering & Testing, Inc.

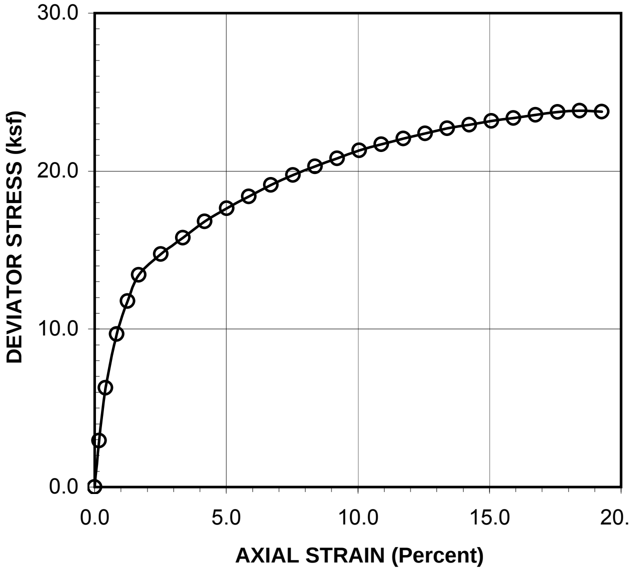
CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

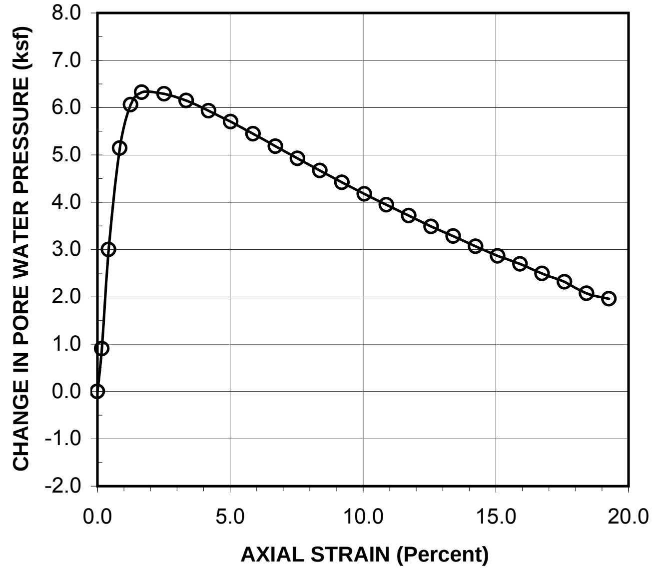
Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	105.0	psi
Project No:	N/A	Back Pressure :	30.0	psi
Boring No.:	MB-1	Consolidation Pressure :	75.0	psi
Depth(ft):	98-99.5	Initial Sample Height:	6.000	in
Sample No.:	SH-11	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.969	in
Sample Description:	Sandy Clay	Final Sample Area (A)*:	6.251	sq. in.
		Induced OCR=	1.0	

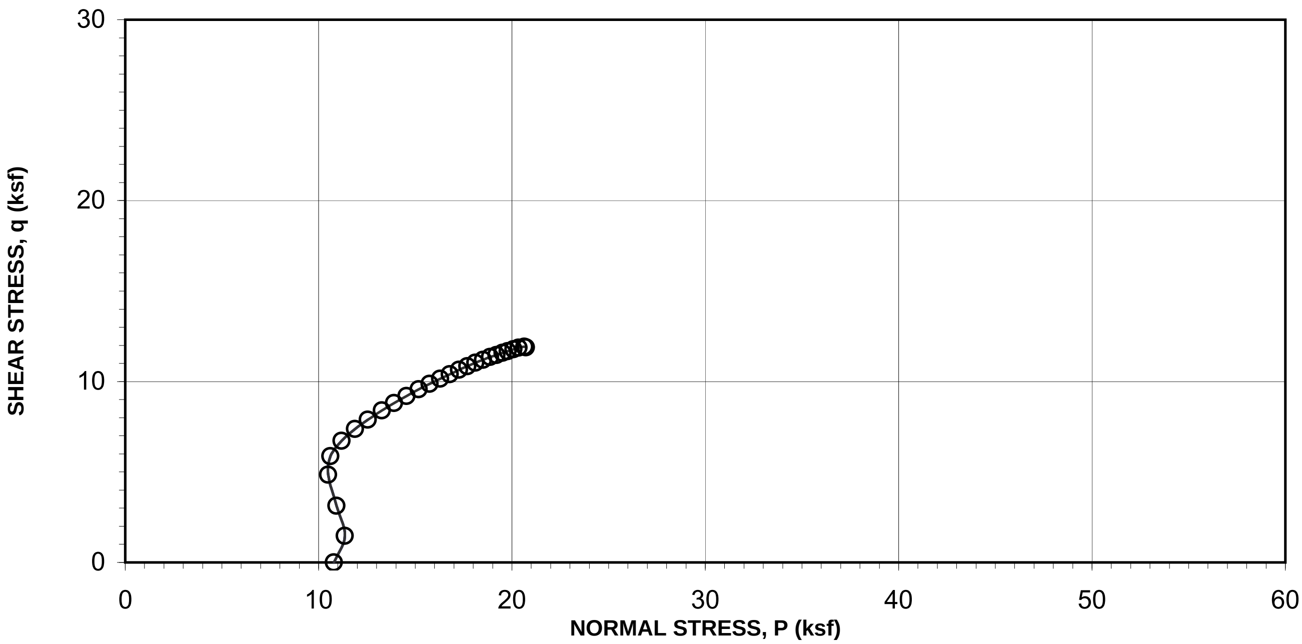
Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
105.0	0	0.000	30.0	0.00	0.00	0.00	0.00	10.80
105.0	128	0.010	36.3	2.94	0.17	0.91	1.47	11.36
105.0	273	0.025	50.8	6.27	0.42	3.00	3.14	10.94
105.0	424	0.050	65.7	9.69	0.84	5.14	4.84	10.50
105.0	517	0.075	72.1	11.76	1.26	6.06	5.88	10.62
105.0	593	0.100	73.9	13.43	1.68	6.32	6.72	11.19
105.0	657	0.150	73.7	14.76	2.51	6.29	7.38	11.88
105.0	709	0.200	72.7	15.79	3.35	6.15	7.89	12.54
105.0	762	0.250	71.2	16.82	4.19	5.93	8.41	13.28
105.0	806	0.300	69.6	17.64	5.03	5.70	8.82	13.92
105.0	848	0.350	67.8	18.39	5.86	5.44	9.20	14.55
105.0	890	0.400	66.0	19.13	6.70	5.18	9.56	15.18
105.0	927	0.450	64.2	19.75	7.54	4.92	9.87	15.75
105.0	962	0.500	62.4	20.31	8.38	4.67	10.15	16.29
105.0	995	0.550	60.7	20.81	9.21	4.42	10.41	16.78
105.0	1028	0.600	59.0	21.30	10.05	4.18	10.65	17.28
105.0	1057	0.650	57.4	21.70	10.89	3.95	10.85	17.70
105.0	1085	0.700	55.8	22.06	11.73	3.72	11.03	18.12
105.0	1111	0.750	54.2	22.38	12.56	3.48	11.19	18.50
105.0	1138	0.800	52.8	22.70	13.40	3.28	11.35	18.87
105.0	1161	0.850	51.3	22.94	14.24	3.07	11.47	19.20
105.0	1184	0.900	49.9	23.16	15.08	2.87	11.58	19.52
105.0	1206	0.950	48.7	23.36	15.92	2.69	11.68	19.79
105.0	1228	1.000	47.3	23.55	16.75	2.49	11.78	20.08
105.0	1250	1.050	46.1	23.73	17.59	2.32	11.87	20.35
105.0	1268	1.100	44.4	23.83	18.43	2.07	11.91	20.64
105.0	1278	1.150	43.6	23.77	19.27	1.96	11.89	20.73

AP Engineering & Testing, Inc.





LEGEND: CONFINING PRESSURE= ○ 10.8 KSF

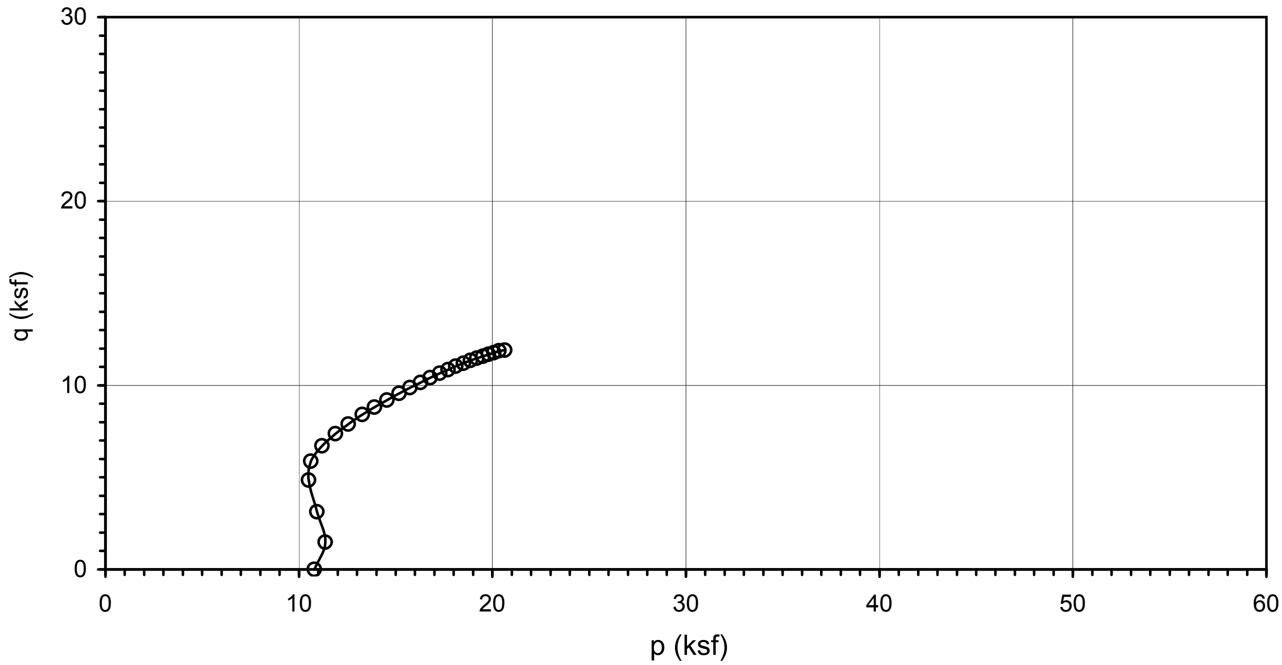


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	123.6
Sample No.:	SH-11	Initial Moisture Content (%):	10.0
Depth (ft):	98-99.5	Confining Pressure:	10.8 ksf

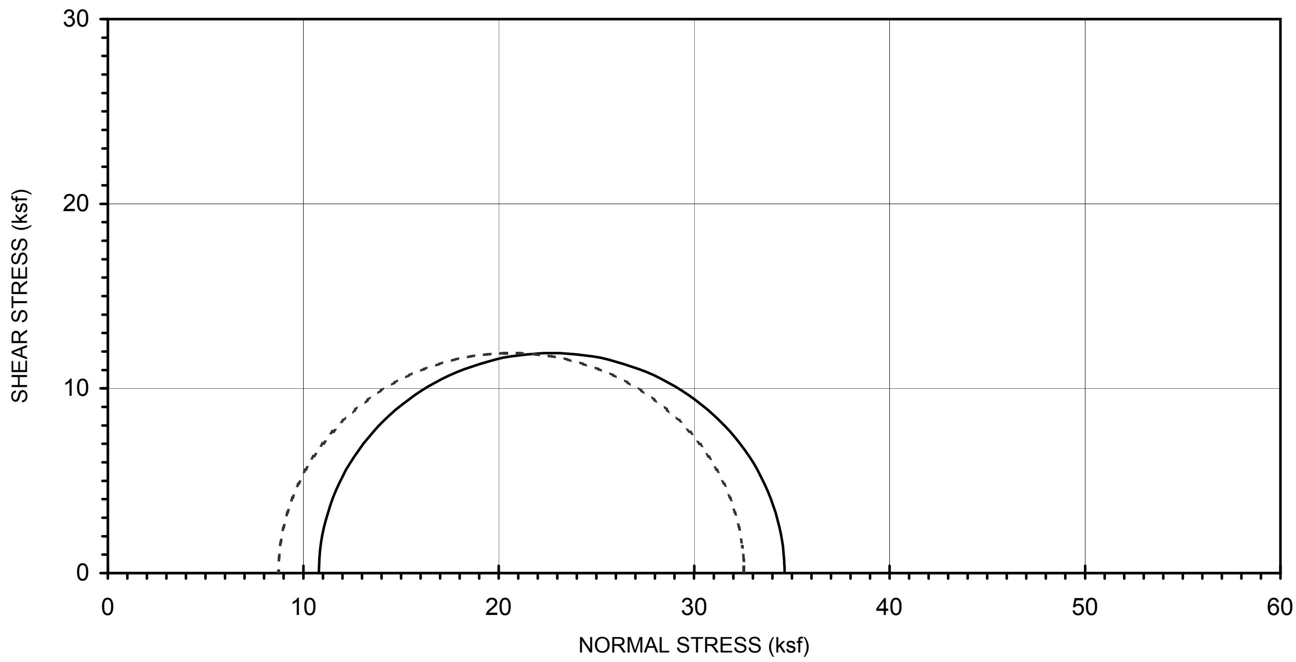
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 10.8 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	123.6
Sample No.:	SH-11	Initial Moisture Content (%):	10.0
Depth (ft):	98-99.5	Confining Pressure:	10.8 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT
ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by: KK	Date: 10-29-08
Project No.:	N/A	Input Data by: KM	Date: 11-06-08
Boring No.:	MB-1	Reviewed by: AP	Date: 11-06-08
Sample No.:	SH-14 (B)	Sample Description: Silty Sand	
Depth(ft):	128-129.3		
Test Condition:	Shelby Tube	Confining Pressure = 15.8 ksf	

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	6.000	6.000	6.000	Avg. =	6.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.265
Moisture Content (%)	18.00	17.57
Wet Weight (gms)	143.78	1466.01
Dry Weight (gms)	129.44	1269.87
Container Weight (gms)	49.76	153.27
Density and Saturation		
Wet Weight (gms)	1327.39	
Container Weight (gms)	0.00	
Wet Density (pcf)	129.8	
Dry Density (pcf)	110.0	
Initial Void Ratio	0.531	
% Saturation	91.5	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	96	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	140.0	Initial Burette Ht.(cm)=	67.1
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	39.1
Eff. Consol. Stress (psi) =	110.0	Final Height (in)=	5.945
Induced OCR =	1.0	Initial Volume (cu.in)=	38.951
Change in Ht. of Specimen (in) =	0.0550	Final Volume (cu.in) =	37.242

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	34.00
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	12.84
Failure Criteria:		Eff. Major Principal stress (ksf) =	46.84
Condition at which maximum deviator stress occurs		Axial Strain (%) =	20.19

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	140.0	psi
Project No:	N/A	Back Pressure :	30.0	psi
Boring No.:	MB-1	Consolidation Pressure :	110.0	psi
Depth(ft):	128-129.3	Initial Sample Height:	6.000	in
Sample No.:	SH-14 (B)	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.945	in
Sample Description:	Silty Sand	Final Sample Area (A)*:	6.265	sq. in.
		Induced OCR=	1.0	

Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
140.0	0	0.000	30.0	0.00	0.00	0.00	0.00	15.84
140.0	245	0.010	35.4	5.62	0.17	0.78	2.81	17.87
140.0	377	0.025	42.3	8.63	0.42	1.77	4.31	18.38
140.0	595	0.050	61.0	13.56	0.84	4.46	6.78	18.16
140.0	714	0.075	74.4	16.21	1.26	6.39	8.10	17.55
140.0	780	0.100	80.4	17.63	1.68	7.26	8.81	17.40
140.0	870	0.150	84.8	19.49	2.52	7.89	9.75	17.70
140.0	961	0.200	85.7	21.35	3.36	8.02	10.67	18.49
140.0	1042	0.250	84.8	22.94	4.21	7.89	11.47	19.42
140.0	1116	0.300	83.2	24.36	5.05	7.66	12.18	20.36
140.0	1194	0.350	80.8	25.83	5.89	7.32	12.92	21.44
140.0	1260	0.400	78.9	27.01	6.73	7.04	13.51	22.31
140.0	1329	0.450	76.3	28.24	7.57	6.67	14.12	23.29
140.0	1390	0.500	73.9	29.26	8.41	6.32	14.63	24.15
140.0	1447	0.550	71.4	30.18	9.25	5.96	15.09	24.97
140.0	1503	0.600	68.6	31.06	10.09	5.56	15.53	25.81
140.0	1538	0.650	66.9	31.49	10.93	5.31	15.74	26.27
140.0	1587	0.700	65.3	32.18	11.77	5.08	16.09	26.85
140.0	1622	0.750	63.7	32.58	12.62	4.85	16.29	27.28
140.0	1661	0.800	62.1	33.04	13.46	4.62	16.52	27.74
140.0	1697	0.850	60.4	33.43	14.30	4.38	16.72	28.18
140.0	1722	0.900	59.3	33.59	15.14	4.22	16.80	28.42
140.0	1749	0.950	58.1	33.78	15.98	4.05	16.89	28.68
140.0	1776	1.000	56.8	33.96	16.82	3.86	16.98	28.96
140.0	1780	1.050	54.7	33.69	17.66	3.56	16.84	29.13
140.0	1810	1.100	53.1	33.90	18.50	3.33	16.95	29.46
140.0	1828	1.150	52.1	33.90	19.34	3.18	16.95	29.61
140.0	1853	1.200	50.8	34.00	20.19	3.00	17.00	29.84

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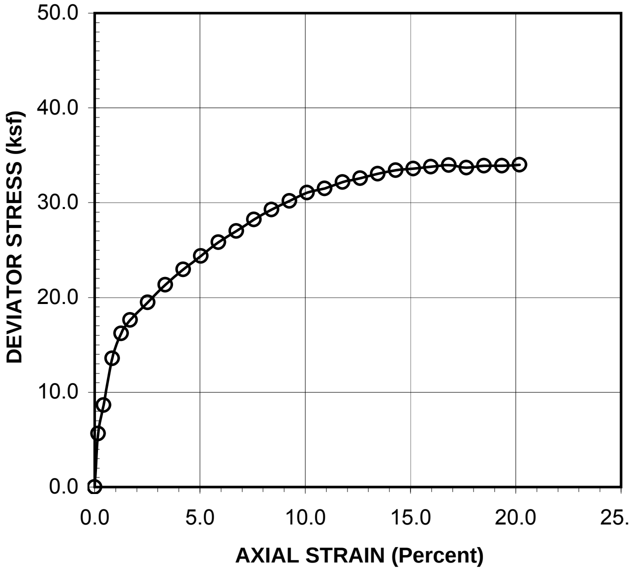
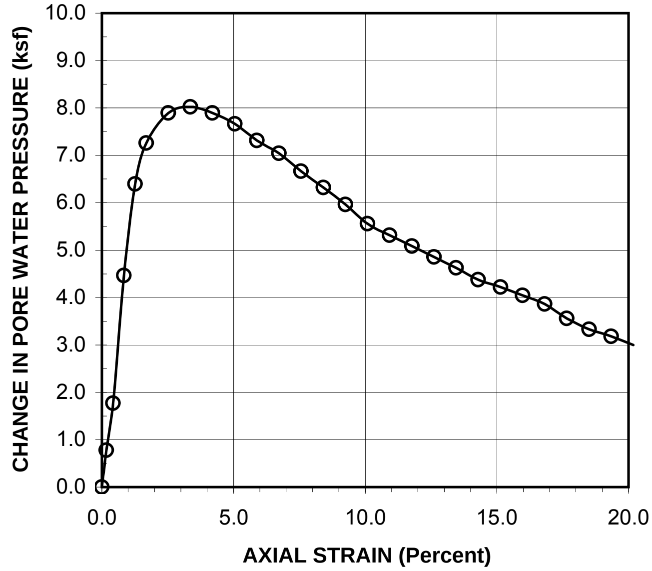
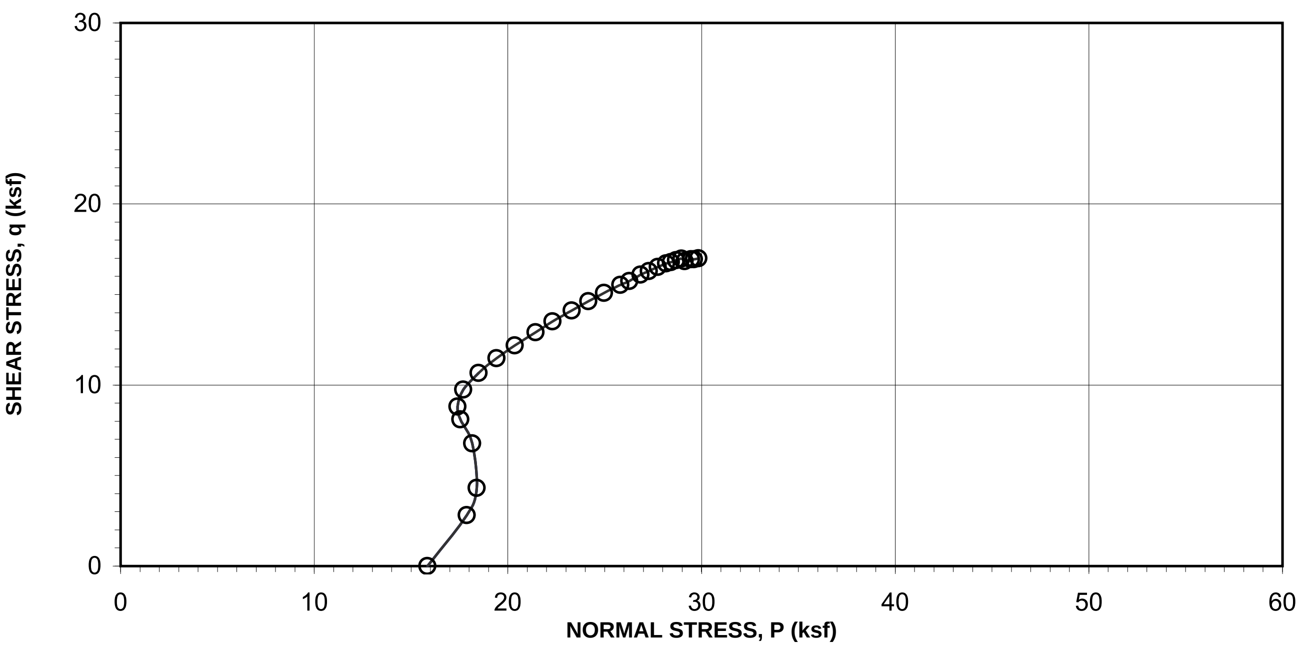




LEGEND: CONFINING PRESSURE= ○ 15.8 KSF

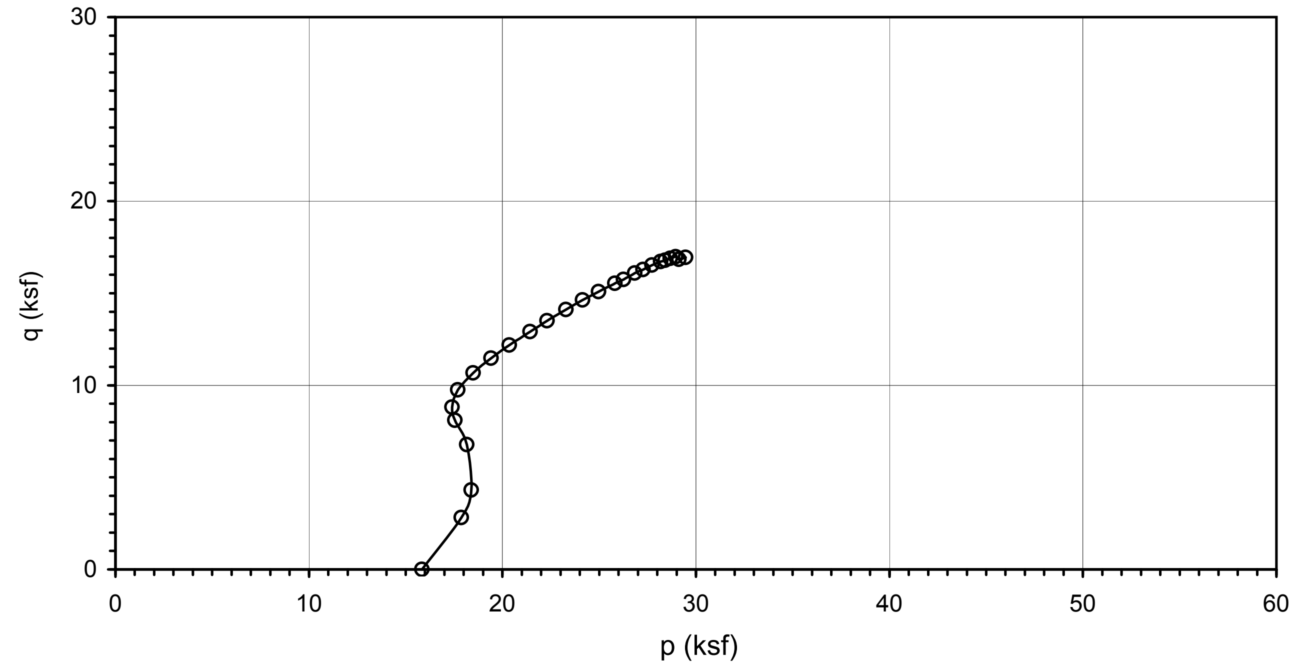


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Silty Sand
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	110.0
Sample No.:	SH-14 (B)	Initial Moisture Content (%):	18.0
Depth (ft):	128-129.3	Confining Pressure:	15.8 ksf

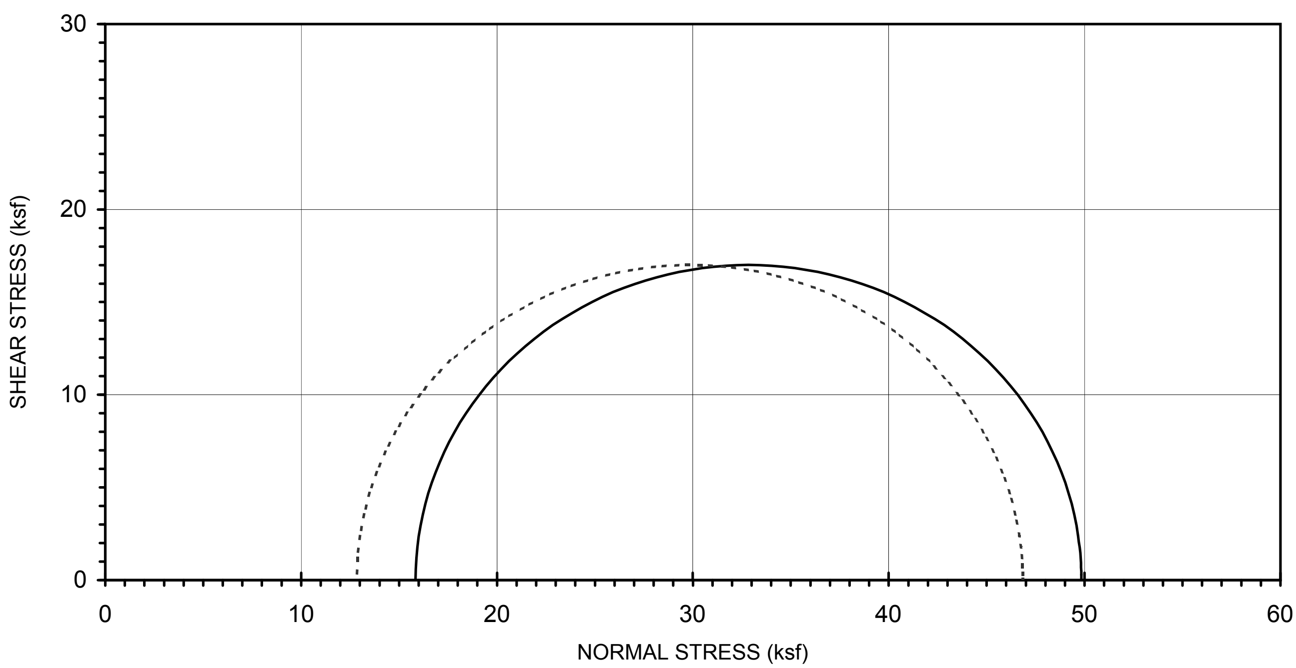
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 15.8 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Silty Sand
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	110.0
Sample No.:	SH-14 (B)	Initial Moisture Content (%):	18.0
Depth (ft):	128-129.3	Confining Pressure:	15.8 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by: KK	Date: 10-22-08
Project No.:	N/A	Input Data by: KM	Date: 10-27-08
Boring No.:	MB-1	Reviewed by: AP	Date: 10-27-08
Sample No.:	SH-14	Sample Description: Sandy Silt	
Depth(ft):	128-129.3		
Test Condition:	Shelby Tube	Confining Pressure = 12.2 ksf	

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	5.000	5.000	5.000	Avg. =	5.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.357
Moisture Content (%)	12.66	17.05
Wet Weight (gms)	88.50	1296.15
Dry Weight (gms)	84.10	1135.63
Container Weight (gms)	49.35	193.95
Density and Saturation		
Wet Weight (gms)	1119.57	
Container Weight (gms)	0.00	
Wet Density (pcf)	131.4	
Dry Density (pcf)	116.6	
Initial Void Ratio	0.445	
% Saturation	76.9	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	96	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	115.0	Initial Burette Ht.(cm)=	55.0
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	36.1
Eff. Consol. Stress (psi) =	85.0	Final Height (in)=	4.925
Induced OCR =	1.0	Initial Volume (cu.in)=	32.459
Change in Ht. of Specimen (in) =	0.0750	Final Volume (cu.in) =	31.306

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	32.92
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	12.23
Failure Criteria:		Eff. Major Principal stress (ksf) =	45.14
Condition at which maximum deviator stress occurs		Axial Strain (%) =	22.34

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	115.0	psi
Project No:	N/A	Back Pressure :	30.0	psi
Boring No.:	MB-1	Consolidation Pressure :	85.0	psi
Depth(ft):	128-129.3	Initial Sample Height:	5.000	in
Sample No.:	SH-14	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	4.925	in
Sample Description:	Sandy Silt	Final Sample Area (A)*:	6.357	sq. in.
		Induced OCR=	1.0	

Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
115.0	0	0.000	30.0	0.00	0.00	0.00	0.00	12.24
115.0	266	0.010	35.6	6.01	0.20	0.81	3.01	14.44
115.0	421	0.025	45.3	9.50	0.51	2.20	4.75	14.79
115.0	544	0.050	59.7	12.20	1.02	4.28	6.10	14.06
115.0	638	0.075	65.1	14.23	1.52	5.05	7.12	14.30
115.0	727	0.100	67.1	16.13	2.03	5.34	8.07	14.97
115.0	879	0.150	66.6	19.31	3.05	5.27	9.65	16.62
115.0	1019	0.200	63.3	22.15	4.06	4.80	11.07	18.52
115.0	1152	0.250	59.2	24.77	5.08	4.20	12.39	20.42
115.0	1270	0.300	54.5	27.02	6.09	3.53	13.51	22.22
115.0	1369	0.350	50.3	28.81	7.11	2.92	14.40	23.72
115.0	1454	0.400	46.3	30.26	8.12	2.35	15.13	25.02
115.0	1530	0.450	42.6	31.49	9.14	1.81	15.75	26.17
115.0	1590	0.500	39.4	32.36	10.15	1.35	16.18	27.07
115.0	1623	0.550	36.5	32.66	11.17	0.94	16.33	27.63
115.0	1635	0.600	34.7	32.53	12.18	0.68	16.26	27.83
115.0	1644	0.650	34.1	32.33	13.20	0.59	16.16	27.81
115.0	1666	0.700	33.7	32.38	14.21	0.53	16.19	27.90
115.0	1686	0.750	33.3	32.38	15.23	0.48	16.19	27.95
115.0	1706	0.800	33.0	32.37	16.24	0.43	16.18	27.99
115.0	1730	0.850	32.8	32.43	17.26	0.40	16.21	28.05
115.0	1762	0.900	32.5	32.62	18.27	0.36	16.31	28.19
115.0	1791	0.950	32.0	32.75	19.29	0.29	16.37	28.33
115.0	1820	1.000	31.4	32.86	20.30	0.20	16.43	28.47
115.0	1846	1.050	30.7	32.90	21.32	0.10	16.45	28.59
115.0	1871	1.100	30.1	32.92	22.34	0.01	16.46	28.68
115.0	1888	1.150	29.6	32.78	23.35	-0.06	16.39	28.69
115.0	1903	1.200	29.4	32.61	24.37	-0.09	16.30	28.63

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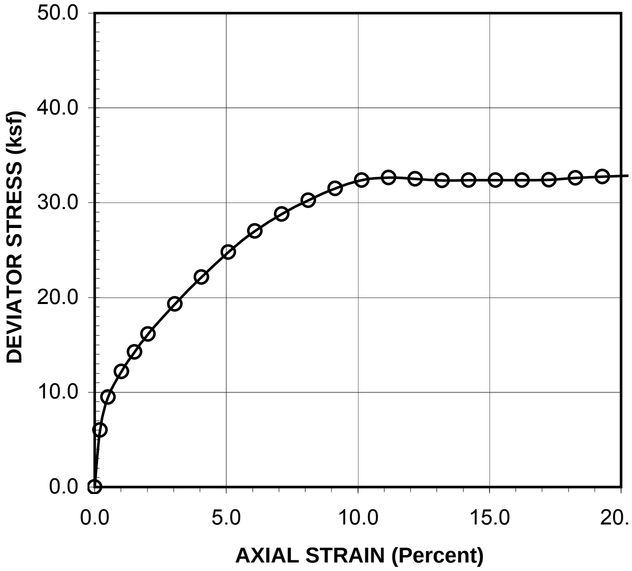
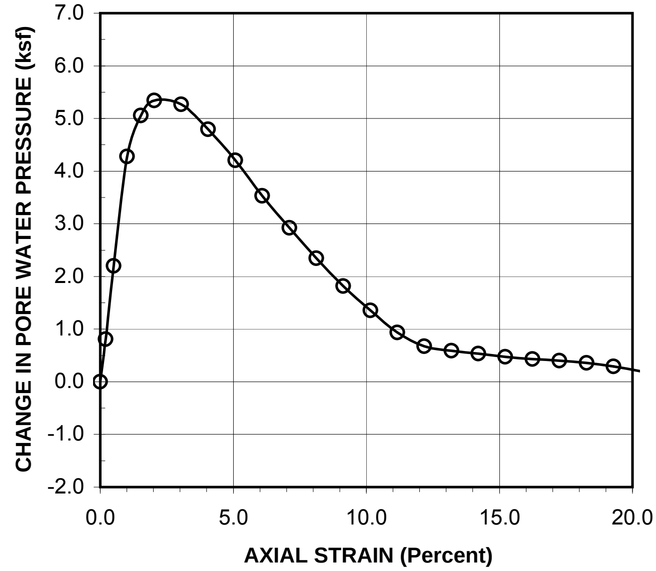
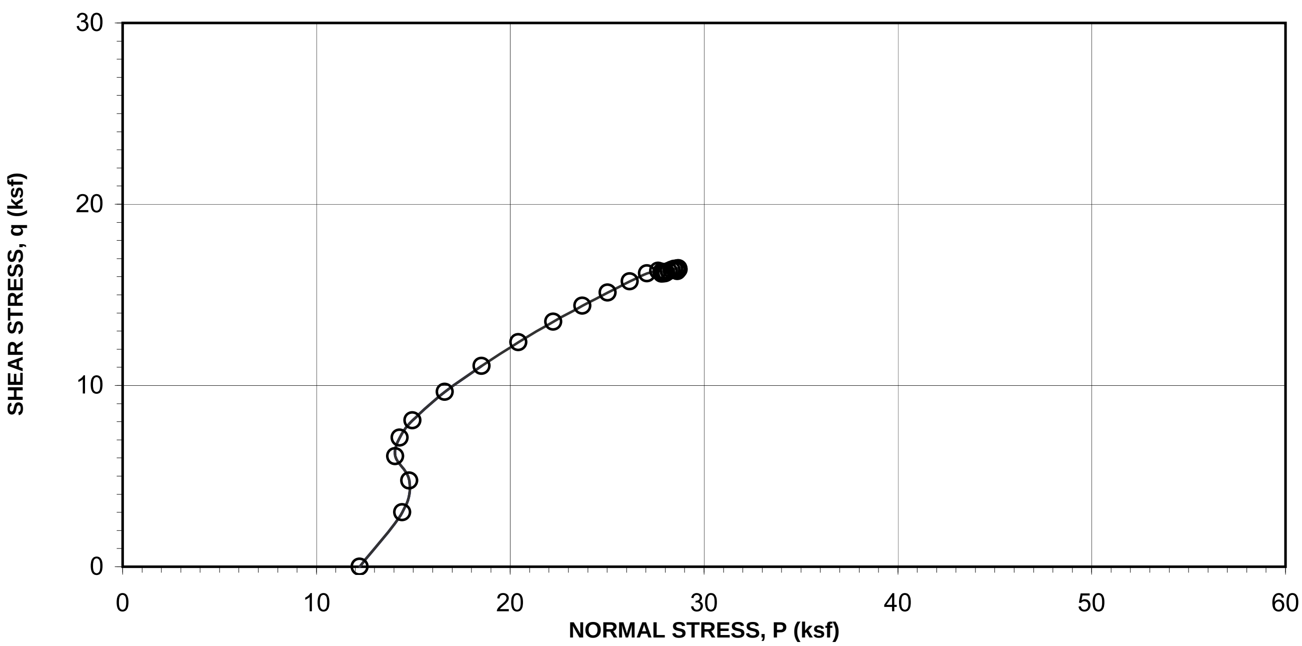




LEGEND: CONFINING PRESSURE= ○ 12.2 KSF

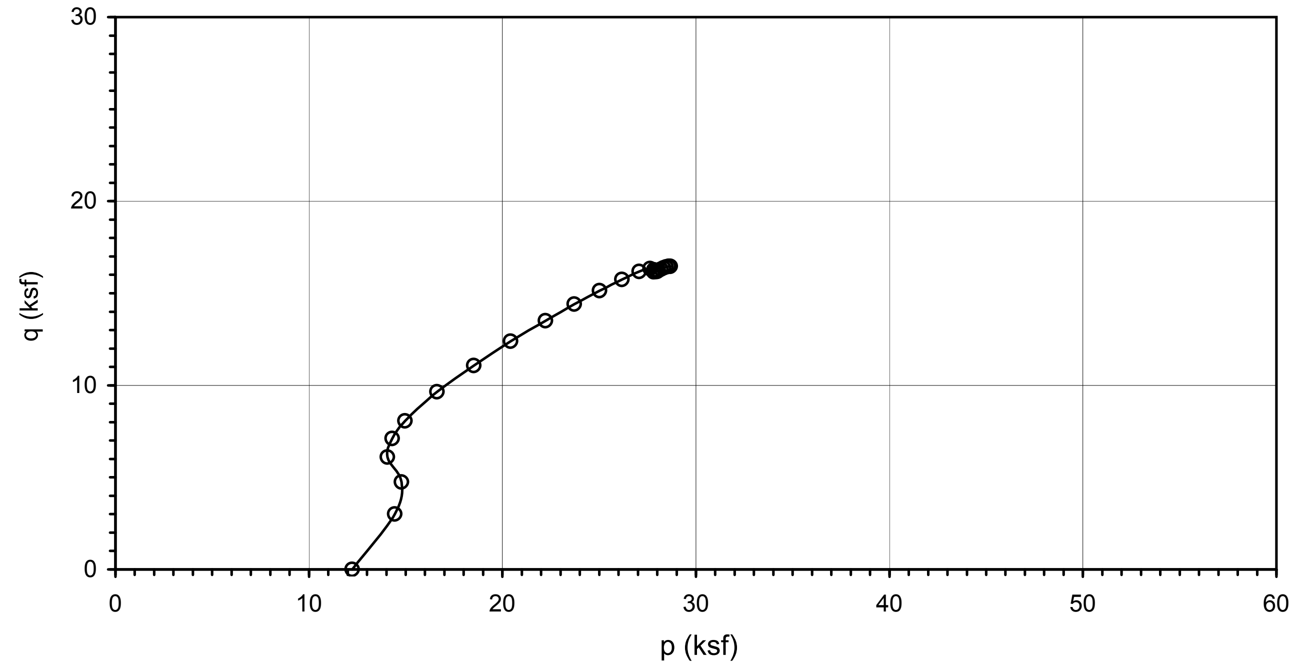


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Silt
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	116.6
Sample No.:	SH-14	Initial Moisture Content (%):	12.7
Depth (ft):	128-129.3	Confining Pressure:	12.2 ksf

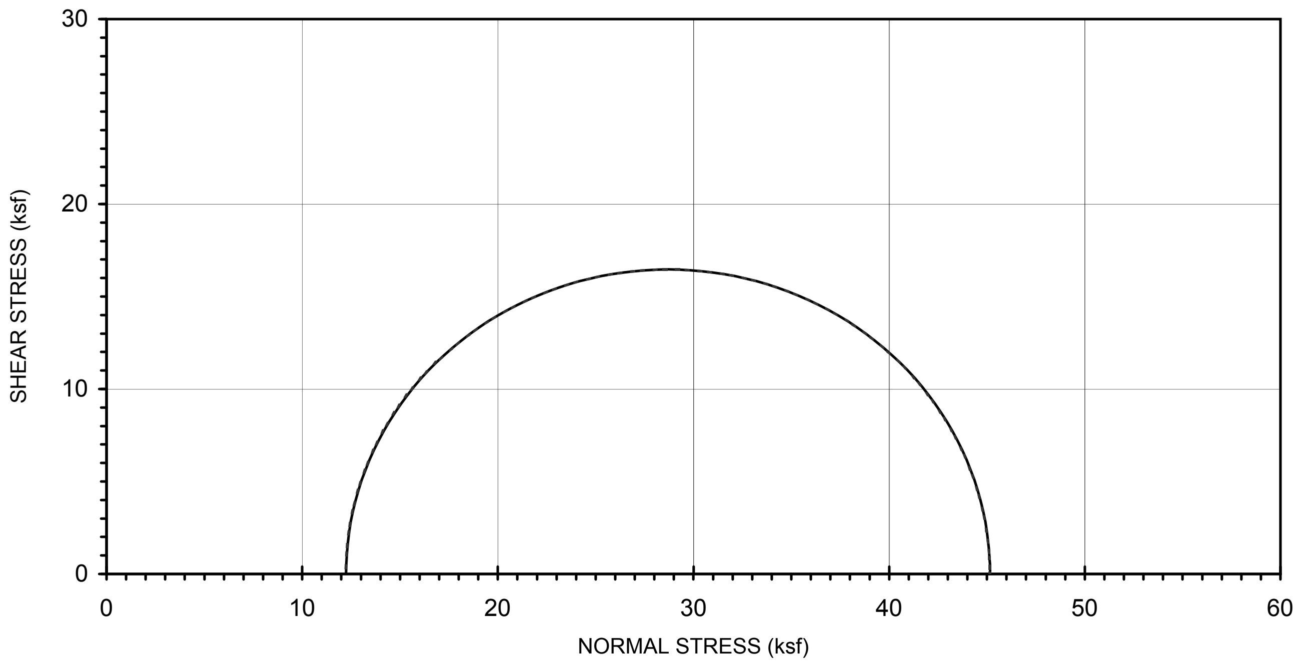
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 12.2 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Silt
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	116.6
Sample No.:	SH-14	Initial Moisture Content (%):	12.7
Depth (ft):	128-129.3	Confining Pressure:	12.2 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT
ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by: KK	Date: 10-21-08
Project No.:	N/A	Input Data by: KM	Date: 10-23-08
Boring No.:	MB-1	Reviewed by: AP	Date: 10-23-08
Sample No.:	SH-15	Sample Description: Sandy Clay	
Depth(ft):	138-139		
Test Condition:	Shelby Tube	Confining Pressure = 13 ksf	

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	6.000	6.000	6.000	Avg. =	6.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.360
Moisture Content (%)	13.51	13.70
Wet Weight (gms)	150.85	1590.44
Dry Weight (gms)	138.96	1422.35
Container Weight (gms)	50.96	195.84
Density and Saturation		
Wet Weight (gms)	1388.87	
Container Weight (gms)	0.00	
Wet Density (pcf)	135.8	
Dry Density (pcf)	119.7	
Initial Void Ratio	0.408	
% Saturation	89.4	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	98	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	120.0	Initial Burette Ht.(cm)=	60.0
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	40.5
Eff. Consol. Stress (psi) =	90.0	Final Height (in)=	5.937
Induced OCR =	1.0	Initial Volume (cu.in)=	38.951
Change in Ht. of Specimen (in) =	0.0630	Final Volume (cu.in) =	37.761

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	43.91
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	13.16
Failure Criteria:		Eff. Major Principal stress (ksf) =	57.08
Condition at which maximum deviator stress occurs		Axial Strain (%) =	8.42

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	120.0	psi
Project No:	N/A	Back Pressure :	30.0	psi
Boring No.:	MB-1	Consolidation Pressure :	90.0	psi
Depth(ft):	138-139	Initial Sample Height:	6.000	in
Sample No.:	SH-15	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.937	in
Sample Description:	Sandy Clay	Final Sample Area (A)*:	6.360	sq. in.
		Induced OCR=	1.0	

Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
120.0	0	0.000	30.0	0.00	0.00	0.00	0.00	12.96
120.0	250	0.010	39.9	5.65	0.17	1.43	2.83	14.36
120.0	388	0.025	50.7	8.75	0.42	2.98	4.37	14.35
120.0	633	0.050	69.8	14.21	0.84	5.73	7.11	14.33
120.0	779	0.075	74.4	17.41	1.26	6.39	8.71	15.27
120.0	915	0.100	75.3	20.37	1.68	6.52	10.18	16.62
120.0	1154	0.150	72.5	25.47	2.53	6.12	12.73	19.57
120.0	1372	0.200	69.0	30.02	3.37	5.62	15.01	22.35
120.0	1553	0.250	63.4	33.68	4.21	4.81	16.84	24.99
120.0	1712	0.300	55.5	36.80	5.05	3.67	18.40	27.69
120.0	1844	0.350	48.3	39.29	5.90	2.64	19.64	29.97
120.0	1964	0.400	41.3	41.47	6.74	1.63	20.73	32.07
120.0	2060	0.450	34.2	43.10	7.58	0.60	21.55	33.91
120.0	2118	0.500	28.6	43.91	8.42	-0.20	21.96	35.12
120.0	2118	0.550	23.5	43.51	9.26	-0.94	21.76	35.65
120.0	2126	0.600	22.4	43.27	10.11	-1.09	21.63	35.69
120.0	2112	0.650	21.1	42.58	10.95	-1.28	21.29	35.53
120.0	2118	0.700	19.9	42.30	11.79	-1.45	21.15	35.56
120.0	2111	0.750	19.4	41.76	12.63	-1.53	20.88	35.36
120.0	2114	0.800	18.9	41.41	13.47	-1.60	20.71	35.26
120.0	2122	0.850	18.0	41.16	14.32	-1.73	20.58	35.27
120.0	2125	0.900	18.2	40.82	15.16	-1.70	20.41	35.07
120.0	2118	0.950	18.4	40.28	16.00	-1.67	20.14	34.77
120.0	2145	1.000	19.1	40.38	16.84	-1.57	20.19	34.72
120.0	2147	1.050	19.6	40.01	17.69	-1.50	20.01	34.47
120.0	2152	1.100	19.9	39.70	18.53	-1.45	19.85	34.26
120.0	2152	1.150	20.0	39.28	19.37	-1.44	19.64	34.04
120.0	2145	1.200	19.7	38.75	20.21	-1.48	19.37	33.82

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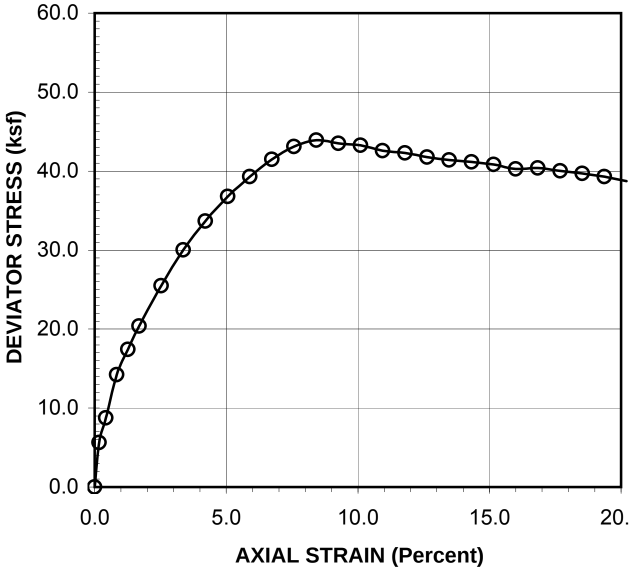
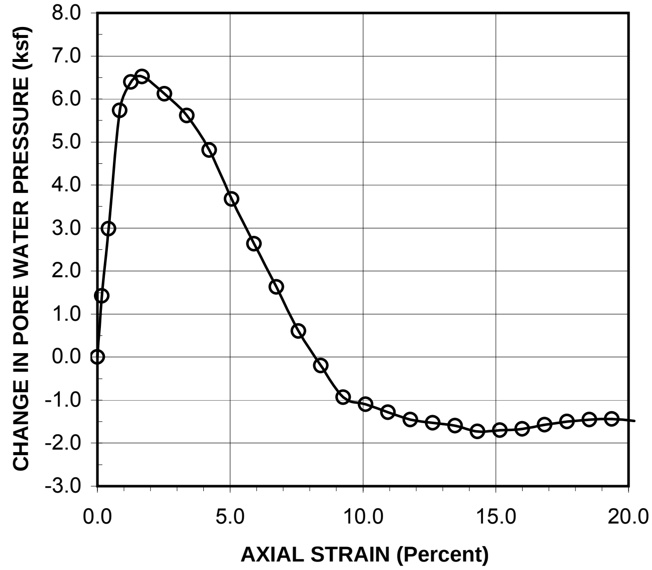
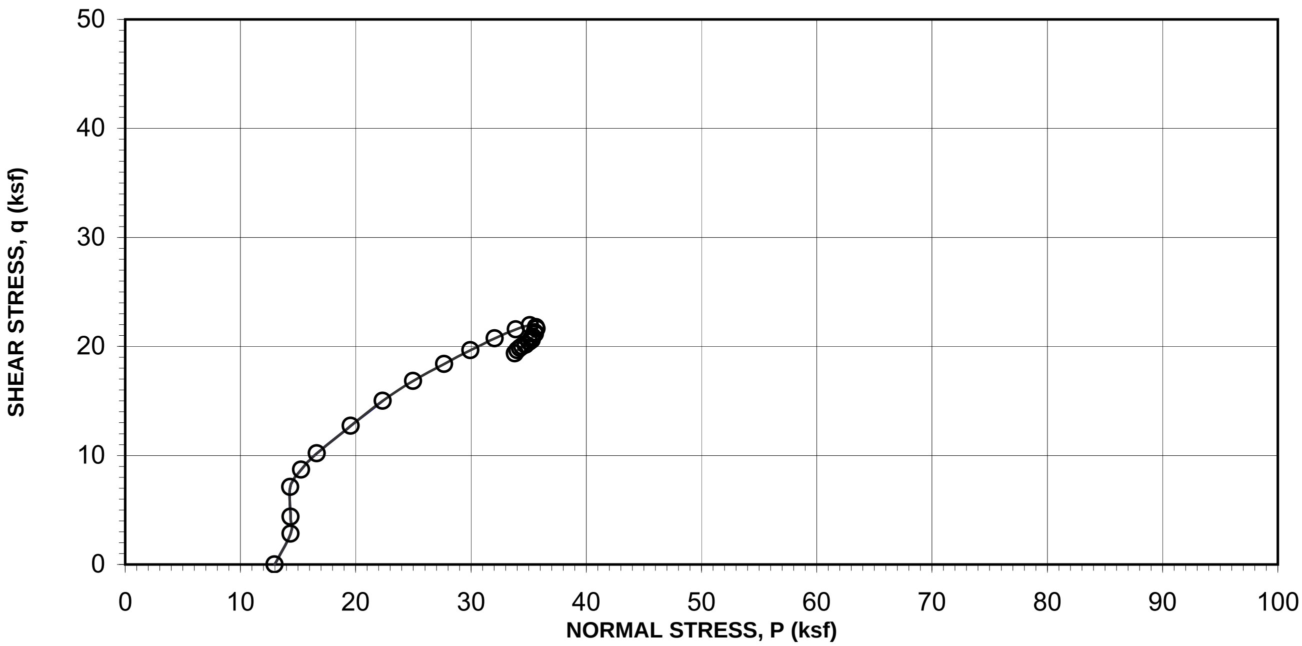




LEGEND: CONFINING PRESSURE= ○ 13 KSF

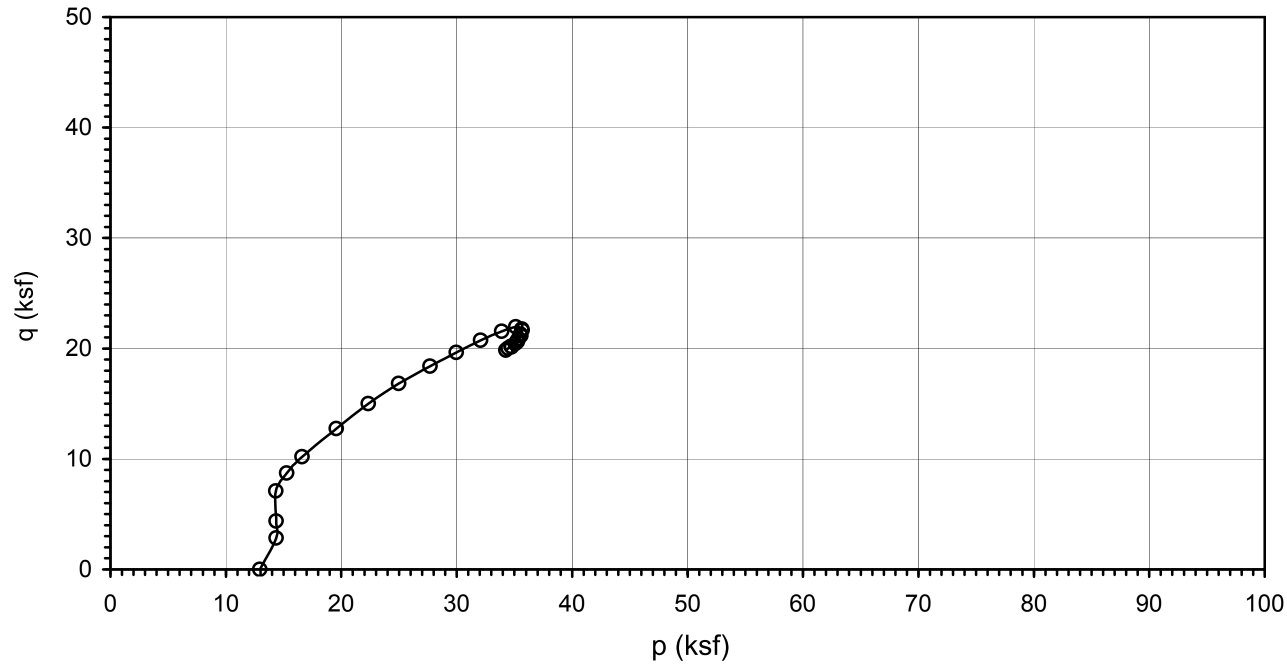


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	119.7
Sample No.:	SH-15	Initial Moisture Content (%):	13.5
Depth (ft):	138-139	Confining Pressure:	13 ksf

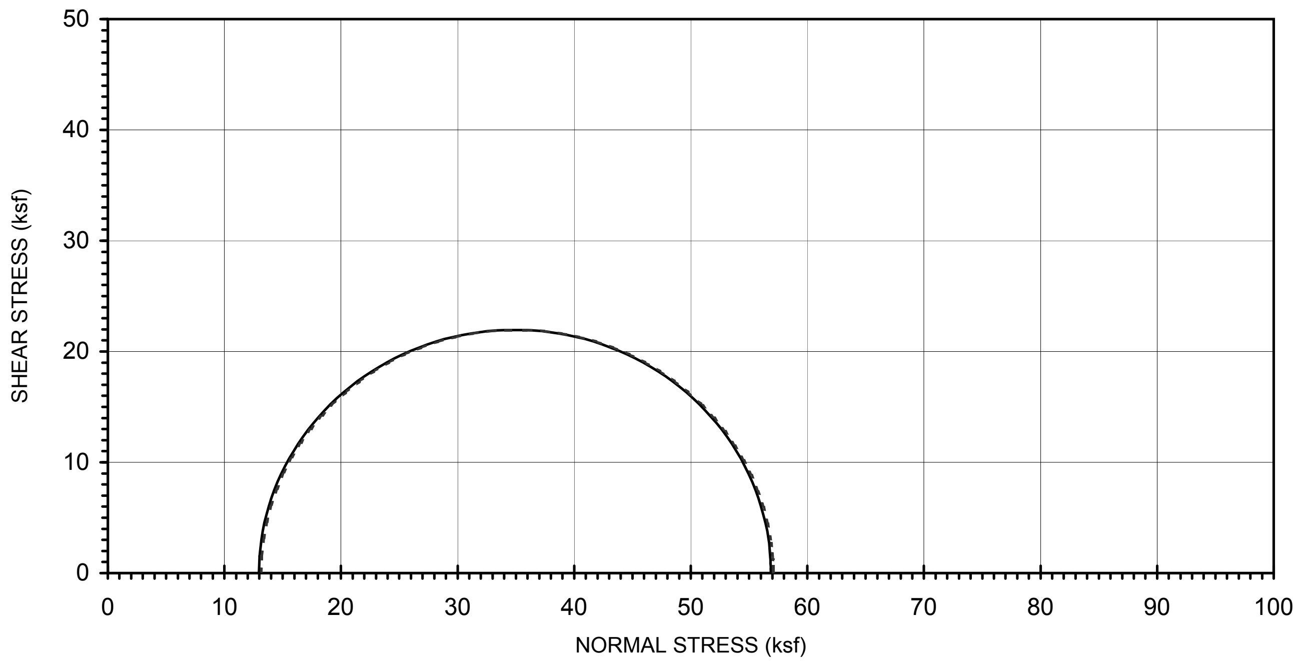
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 13 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	119.7
Sample No.:	SH-15	Initial Moisture Content (%):	13.5
Depth (ft):	138-139	Confining Pressure:	13 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	Camanche Embankments	Tested by: KK	Date: 10-21-08
Project No.:	N/A	Input Data by: KM	Date: 10-23-08
Boring No.:	MB-1	Reviewed by: AP	Date: 10-23-08
Sample No.:	SH-19	Sample Description: Sandy Clay	
Depth(ft):	178-178.8		
Test Condition:	Shelby Tube	Confining Pressure = 15.8 ksf	

Diameter (in)	2.875	2.875	2.875	Avg. =	2.875
Height (in)	6.000	6.000	6.000	Avg. =	6.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	6.492	6.319
Moisture Content (%)	13.28	14.25
Wet Weight (gms)	142.37	1537.12
Dry Weight (gms)	131.53	1369.46
Container Weight (gms)	49.89	193.13
Density and Saturation		
Wet Weight (gms)	1351.32	
Container Weight (gms)	0.00	
Wet Density (pcf)	132.2	
Dry Density (pcf)	116.7	
Initial Void Ratio	0.444	
% Saturation	80.7	

Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	95	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	140.0	Initial Burette Ht.(cm)=	49.9
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	23.6
Eff. Consol. Stress (psi) =	110.0	Final Height (in)=	5.910
Induced OCR =	1.0	Initial Volume (cu.in)=	38.951
Change in Ht. of Specimen (in) =	0.0900	Final Volume (cu.in) =	37.346

Shear		At Failure	
Rate of Deformation (in/min)=	0.0050	Deviator Stress (ksf) =	39.85
Time to 50% primary Consolidation =	min.	Eff. Minor Principal stress (ksf) =	14.56
Failure Criteria:		Eff. Major Principal stress (ksf) =	54.40
Condition at which maximum deviator stress occurs		Axial Strain (%) =	19.46

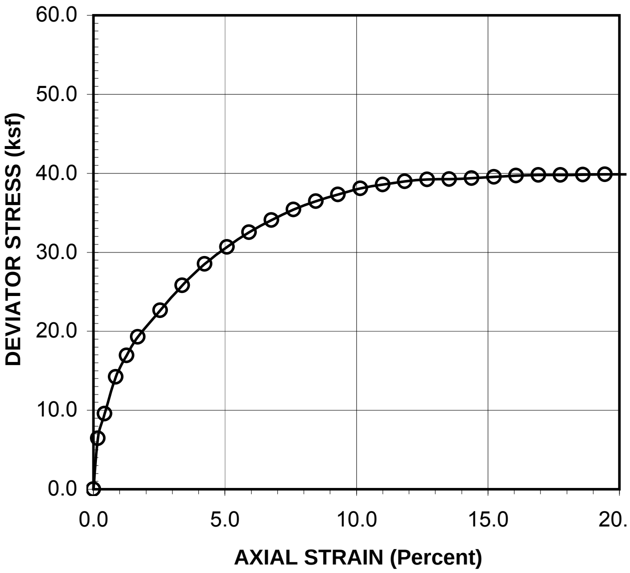
CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

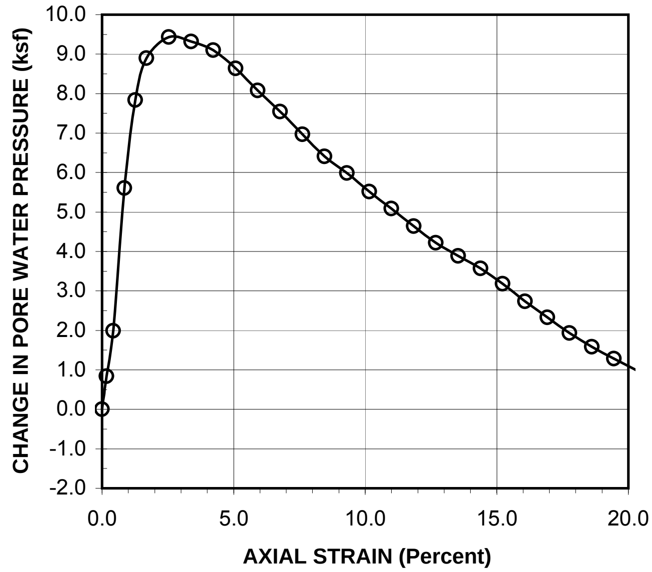
Cell No. 1

Project Name:	Camanche Embankments	Cell Pressure:	140.0	psi
Project No:	N/A	Back Pressure :	30.0	psi
Boring No.:	MB-1	Consolidation Pressure :	110.0	psi
Depth(ft):	178-178.8	Initial Sample Height:	6.000	in
Sample No.:	SH-19	Initial Area of Sample:	6.492	sq. in.
Sample Type:	Shelby Tube	Final Sample Ht.* (L):	5.910	in
Sample Description:	Sandy Clay	Final Sample Area (A)*:	6.319	sq. in.
		Induced OCR=	1.0	

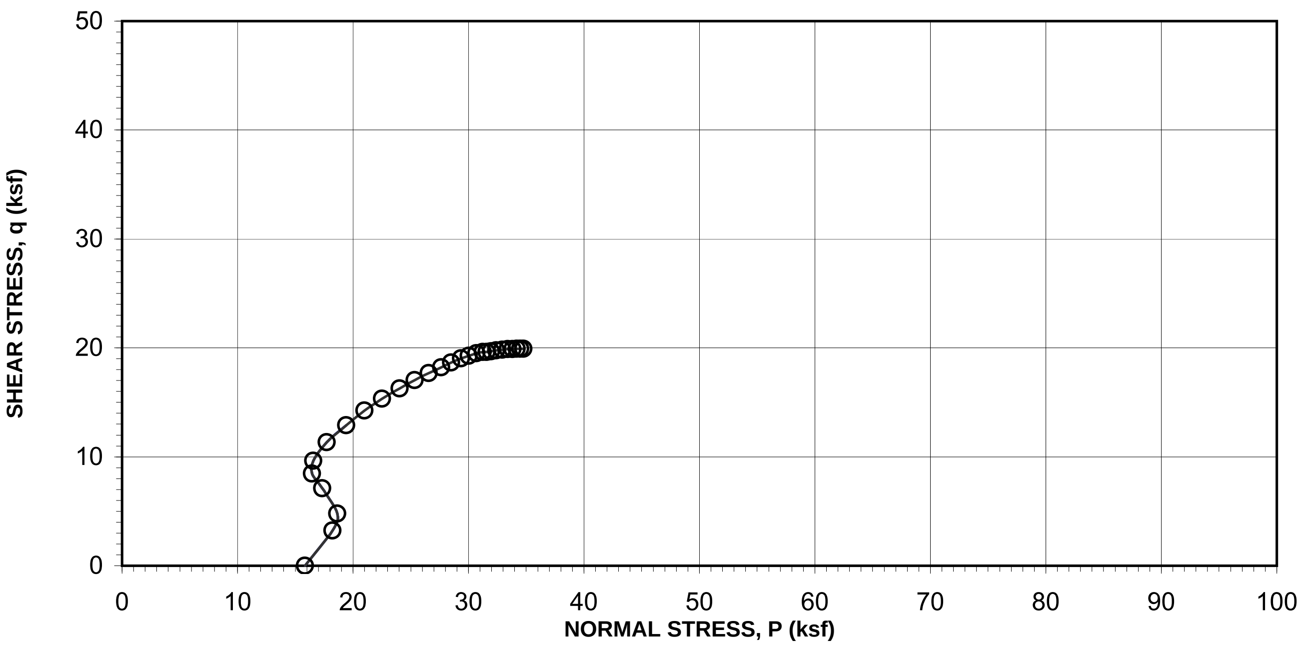
Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure `	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
140.0	0	0.000	30.0	0.00	0.00	0.00	0.00	15.84
140.0	284	0.010	35.8	6.46	0.17	0.84	3.23	18.24
140.0	422	0.025	43.8	9.58	0.42	1.99	4.79	18.64
140.0	629	0.050	68.9	14.21	0.85	5.60	7.11	17.34
140.0	752	0.075	84.4	16.92	1.27	7.83	8.46	16.47
140.0	860	0.100	91.8	19.27	1.69	8.90	9.63	16.57
140.0	1020	0.150	95.5	22.65	2.54	9.43	11.33	17.73
140.0	1172	0.200	94.7	25.80	3.38	9.32	12.90	19.42
140.0	1306	0.250	93.2	28.50	4.23	9.10	14.25	20.99
140.0	1417	0.300	90.0	30.65	5.08	8.64	15.33	22.53
140.0	1518	0.350	86.1	32.54	5.92	8.08	16.27	24.03
140.0	1604	0.400	82.4	34.08	6.77	7.55	17.04	25.33
140.0	1681	0.450	78.4	35.39	7.61	6.97	17.69	26.57
140.0	1747	0.500	74.5	36.44	8.46	6.41	18.22	27.65
140.0	1806	0.550	71.6	37.32	9.31	5.99	18.66	28.51
140.0	1860	0.600	68.3	38.08	10.15	5.52	19.04	29.37
140.0	1901	0.650	65.3	38.56	11.00	5.08	19.28	30.03
140.0	1941	0.700	62.2	38.99	11.84	4.64	19.50	30.70
140.0	1972	0.750	59.3	39.23	12.69	4.22	19.62	31.24
140.0	1992	0.800	57.0	39.25	13.54	3.89	19.62	31.58
140.0	2018	0.850	54.8	39.37	14.38	3.57	19.69	31.95
140.0	2046	0.900	52.1	39.52	15.23	3.18	19.76	32.42
140.0	2075	0.950	49.0	39.68	16.07	2.74	19.84	32.95
140.0	2101	1.000	46.2	39.78	16.92	2.33	19.89	33.40
140.0	2124	1.050	43.4	39.80	17.77	1.93	19.90	33.81
140.0	2148	1.100	41.0	39.84	18.61	1.58	19.92	34.17
140.0	2171	1.150	38.9	39.85	19.46	1.28	19.92	34.48
140.0	2194	1.200	36.9	39.84	20.30	0.99	19.92	34.77

AP Engineering & Testing, Inc.





LEGEND: CONFINING PRESSURE= ○ 15.8 KSF

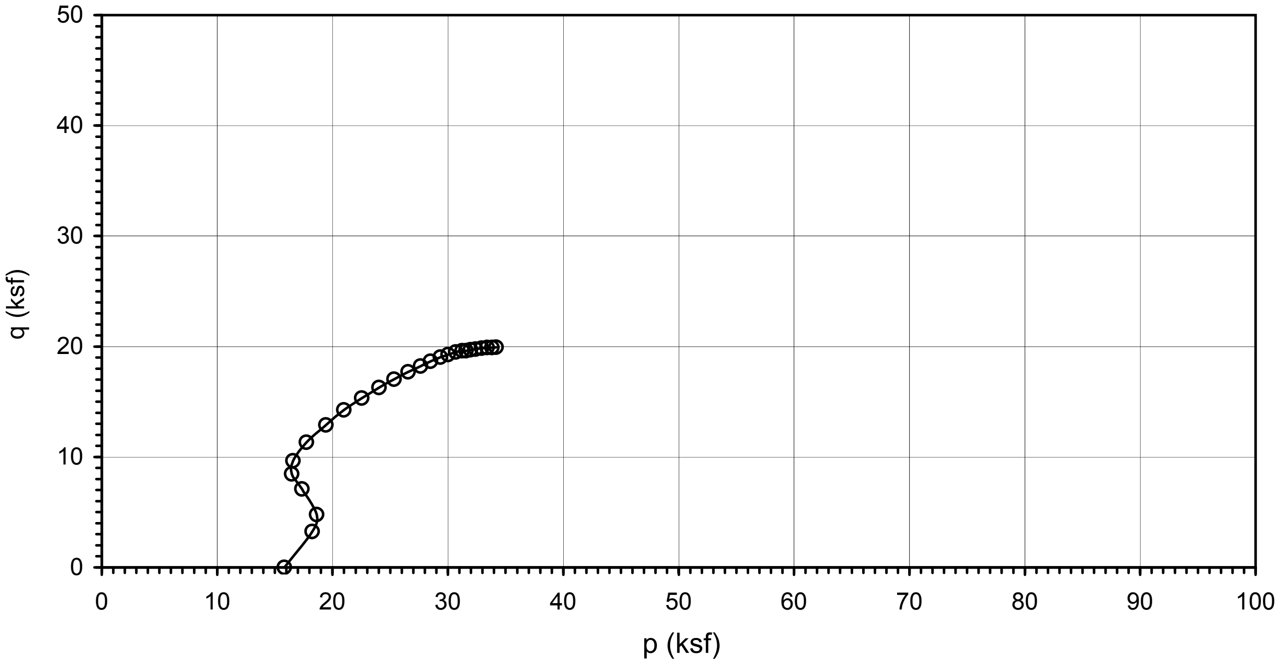


Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	116.7
Sample No.:	SH-19	Initial Moisture Content (%):	13.3
Depth (ft):	178-178.8	Confining Pressure:	15.8 ksf

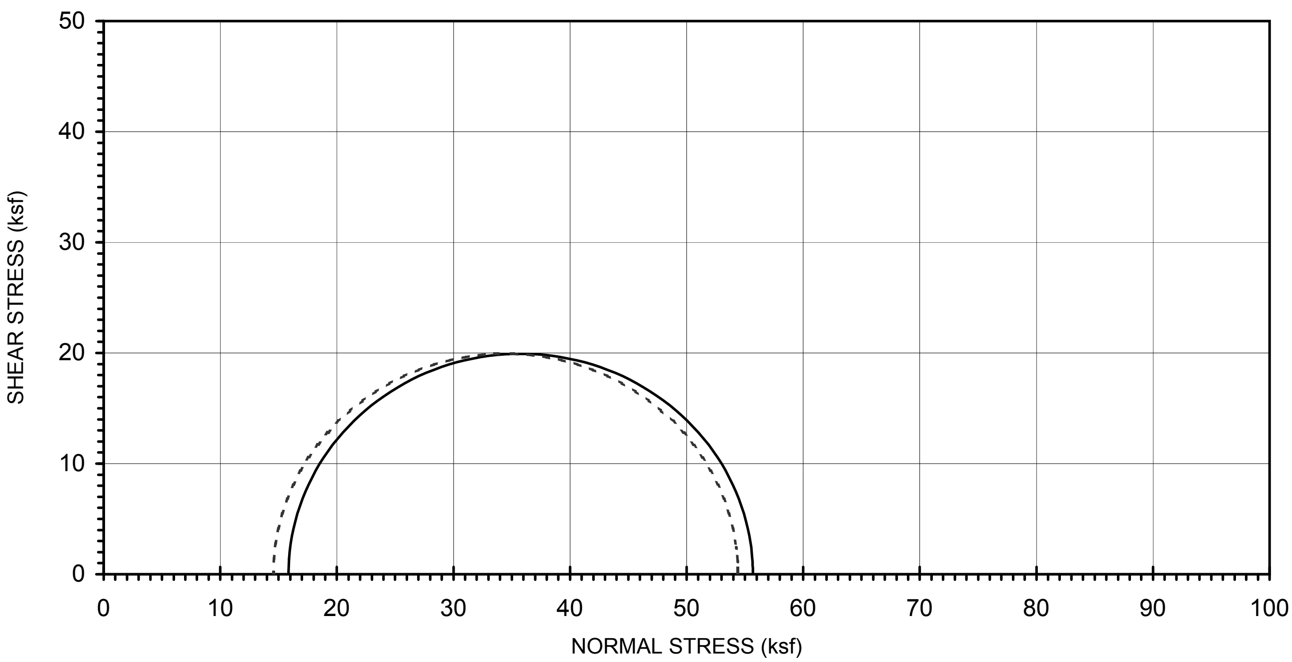
CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

AP Engineering & Testing, Inc.



LEGEND: CONFINING PRESSURE= ○ 15.8 KSF



Project Name:	Camanche Embankments	Sample Type:	Shelby Tube
Project No.:	N/A	Sample Description:	Sandy Clay
Boring No.:	MB-1	Initial Dry Unit Weight (pcf):	116.7
Sample No.:	SH-19	Initial Moisture Content (%):	13.3
Depth (ft):	178-178.8	Confining Pressure:	15.8 ksf

CU TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT
ASTM D 4767

ENCLOSURE D
REPORT FROM
INTERNAL TECHNICAL REVIEW BOARD

CAMANCHE EMBANKMENTS SAFETY REVIEW:
INTERNAL TECHNICAL REVIEW BOARD
MEETING NO. 1

July 8, 2009

Guilaine Roussel, P.E., Principal
Terra Engineers, Inc.
350 Sansome Street, Suite 830
San Francisco, CA 94104

Subject: Camanche Embankments Safety Review: Internal Technical Review Board (ITRB) Meeting No. 1

Dear Ms. Roussel:

This letter report is submitted in response to your request for our review comments regarding the Terra/GeoPentech Joint Venture (T/GP JV) team's findings and conclusions as summarized in the document, "Camanche Embankments Safety Review - Technical Review Board – Background Information for June 2009 Meeting" and presented to us at the June 29-30, 2009 meetings held at the Meridien Hotel in San Francisco (June 29) and East Bay Municipal Utilities District (EBMUD) Office in Oakland (June 30).

The above-referenced document, which was provided to us in advance of the June 29-30 meetings, contained the following technical memoranda:

1. Material Characterization at Main Dam (Draft)
2. Analyses of Seepage and Uplift Pressures (Draft)
3. Maximum Design Earthquake Ground Motions and Time Histories
4. Seismic Deformation of Camanche Reservoir Main Dam (Draft)

We were also provided with copies of the technical presentations made by the T/GP JV team at the meetings.

Copies of the meeting agenda and attendance lists for the two meetings are attached.

This letter report contains our review comments regarding the T/GP JV team's conclusions and findings as presented at the meeting and contained in the above-reference documents.

1. <u>Material Characterization at the Main Dam:</u>

The main dam is a compacted zoned embankment dam with dredged tailings underlying portions of the upstream and downstream shells. The tailings were excavated beneath the central core to extend the core to bedrock. The tailings were also removed from the area along the outlet works trench near the right abutment.

The material characterization at the main dam integrated results from new explorations, previous explorations by Wahler (1981), and knowledge of the construction history and local geology. The new explorations included borings with Standard Penetration Tests (SPT), a series of cone penetration tests (CPT), and laboratory tests on samples obtained from the field. The new SPT data, obtained at the downstream toe of the main dam, were systematically compared to the data previously obtained in that area by Wahler (1981). The two data sets were reasonably consistent, providing a measure of calibration between the older and new data sets. The combined data sets were systematically evaluated for their spatial distributions and consistency with the construction history and local geology. The draft report provides clear documentation of the basis and development of the material characterization.

The main dam is well-compacted and zoned with appropriate transitions between the coarser shell materials (typically gravelly sand to sandy gravel) and finer core materials (typically sandy clay and sandy silt with areas of clayey sand and silty sand). The T/GP JV team indicated at the meeting that the various zone contacts within the dam were checked for, and found to meet, the filter criteria developed by Sherard and colleagues since the dam was constructed. Shear strength parameters and dynamic properties for the core and shells were developed appropriately. We concur with the static and dynamic properties selected by the T/GP JV team for the core and shell materials of the main dam.

The layer of tailings material beneath the upstream and downstream shells is typically 10 to 25 feet thick and comprised of liquefiable sands, silty sands, and gravels with some zones of silt. The characterization data indicate that the tailings are loosest near the upstream and downstream toes and denser near the core under the shells.

The SPT data indicate that the tailings are loose at the downstream and upstream toes, which is consistent with the expected state for young dredged tailing materials. The T/GP JV team selected representative SPT $(N_1)_{60cs}$ values for the toe areas after considering the spatial distributions with respect to depth and position along the dam and considering different percentile values from within those distributions. We similarly examined the SPT data from near the downstream toe for spatial distributions. For example, SPT $(N_1)_{60cs}$ values obtained in 1981 by Wahler and those obtained by T/GP JV team in 2008 in the tailings at the downstream toe are presented in Figure 1 herein.

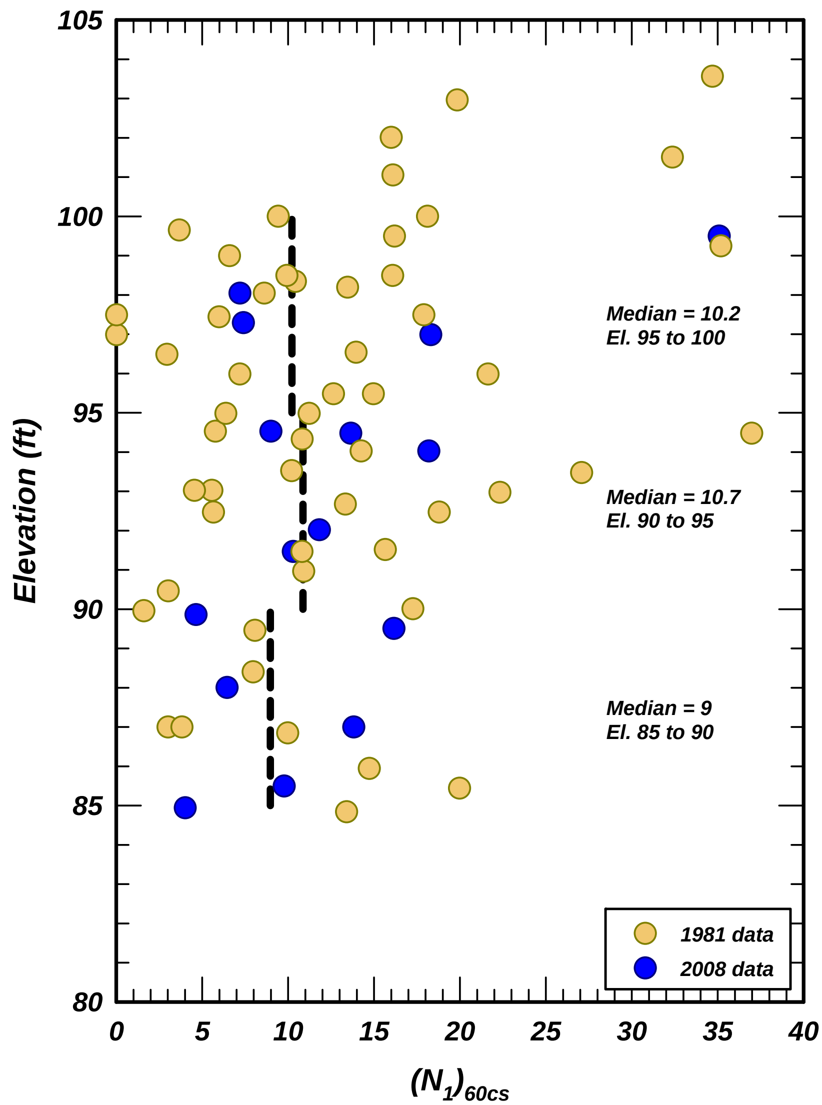


Figure 1 Values of $(N_1)_{60cs}$ obtained in 1981 and 2008 in tailings adjacent to downstream toe

The information provided in Figure 1 indicates that:

a. The values of $(N_1)_{60cs}$ obtained in 1981 are comparable to those obtained in 2008, which supports the use of all the 1981 data;
b. The $(N_1)_{60cs}$ values are slightly smaller at the lower elevations within the tailings, with the data between elevations of 85 and 90 feet being well represented by their median $(N_1)_{60cs}$ value of about 9, as shown in the figure.

The $(N_1)_{60cs}$ data at the higher elevations in the tailings adjacent to the downstream toe include a greater number of somewhat higher blow counts. The median values over specific elevation intervals are listed below:

Elevation Range	$(N_1)_{60cs}$
85 to 90	9
90 to 95	10.7
95 to 100	10.2
85 to 104 (all values)	11.3
85 to 104 [$(N_1)_{60cs} \leq 25$]	10.3

The T/GP JV team considered the upper and lower elevation data together, but they reasonably considered the effects of excluding some of the higher $(N_1)_{60cs}$ values (primarily from the upper elevations), which produces a more uniform distribution versus elevation, as indicated in the listing above.

Ultimately, we conclude that the representative $(N_1)_{60cs}$ values selected by the T/GP JV team, and the range of values covered in their seismic deformation sensitivity analyses, are both reasonable and consistent with the depositional history of these soils, their gradational characteristics, and the supporting CPT data. The median $(N_1)_{60cs}$ value of 9 for the tailings between elevations of 85 and 90 feet adjacent to the downstream toe, as shown in the above figure, falls between the T/GP JV team's selections of a 33rd percentile $(N_1)_{60cs}$ value of 8 and median $(N_1)_{60cs}$ value of 11 for representing the tailings adjacent to the downstream toe. Therefore, we suggest that the seismic deformations for cases where the tailings at the downstream toe are represented by $(N_1)_{60cs} = 8$ or 9 are the most appropriate for evaluating the dam's seismic performance and associated consequences.

The SPT data indicate that the tailings become progressively denser under the shells with increasing distance from the toes; e.g., representative $(N_1)_{60cs}$ values near the core are more than double the values near the toes. This trend in $(N_1)_{60cs}$ values is important to the seismic evaluation and thus it is worthwhile evaluating the extent to which this trend can be physically explained versus being a possible artifact of the SPT testing procedures at large depths (e.g., energy transmission, overburden correction factor). Physically, this trend in $(N_1)_{60cs}$ values may be due to the cumulative effect of several factors, which might include: the re-working of dredge materials during construction of the dam (this effect might be greatest near the core where the tailings are thinnest); compression of the tailings under the weight of the shells; the tailings being over-consolidated when the reservoir is high (pre-consolidation stresses being governed by low reservoir conditions); and possible differences in dredging operations across the dam footprint (perhaps this could be evaluated by checking the regularity of the bedrock profile). We suggest that the reasons for the observed trend in $(N_1)_{60cs}$ values warrants more detailed discussion, with further examination of construction records and other pertinent information to the extent possible. Nonetheless, we recognize that it may ultimately be difficult to quantitatively explain the magnitude of the observed trends in $(N_1)_{60cs}$ values. In this regard, the T/GP JV team's evaluations of the sensitivity of the seismic deformation analysis results (discussed later in this letter) to variations in the trends of the $(N_1)_{60cs}$ values are particularly relevant and insightful.

We examined the relationships developed by the T/GP JV team for expressing how the residual shear strength of liquefied tailings materials would vary with position under the dam. These strengths were based on case history based correlations and the observed trends in $(N_1)_{60cs}$ values. The residual strength correlations by Idriss and Boulanger (2007) were limited to effective overburden stresses of about 4 tsf, beyond which we recommended that the relationships could be adjusted by applying a further correction to the SPT $(N_1)_{60cs}$ values. We believe the residual strength relationships used by the T/GP JV team, subject to the adjustment previously mentioned, are reasonable for evaluating the seismic performance of the main dam.

The possible impact of variations in the material characterizations were later evaluated by the T/GP JV team in the seismic deformation analyses. These sensitivity studies provide insight on what aspects of the material characterization are most important and how uncertainties in the properties affected the predicted performance. Based on the results of those sensitivity analyses, as discussed later, we believe that the material characterization of the main dam provided a well-supported basis for evaluating its seismic performance.

2. <u>Analyses of Seepage and Uplift Pressures:</u>

The analysis methods and procedures used for evaluating seepage and uplift pressures were generally appropriate and reasonable.

The effect of the relief wells on the seepage conditions for the main dam are an important consideration for the evaluation of seismic performance because the potential effects of damage to the relief wells will need to be considered. For this reason, we suggest that the piezometer and reservoir level records from the first filling, before installation of the first set of relief wells, to past the installation of the second set of relief wells, be further examined. Evaluation of these data and the associated head ratios may provide an improved understanding of how damage to the relief wells might impact the seepage conditions and performance of the dam. The results of this evaluation would also provide a basis for evaluating options that involve accepting the risk of liquefaction-induced damage to the relief wells while establishing action plans that specify the nature and urgency of repairs in the event that liquefaction-induced damage occurs.

The analyses of Dike 5 predicted the existence of high exit gradients around the downstream toe. We concur with the T/GP JV team in suggesting that the installation of piezometers at this dike would be an appropriate next step, as they would provide the basis for evaluating whether the predicted seepage conditions are reasonable or if the seepage conditions are better than predicted.

The data for the main dam suggest that the effectiveness of the relief wells has decreased over time, as evidenced by some rising piezometer levels and decreasing relief well flow rates. The T/GP JV team suggested that maintenance of the relief wells would be prudent. We concur with this recommendation, but note that any action can reasonably be delayed until it can be implemented in coordination with any actions that are taken in response to seismic performance issues.

While the seepage analysis results appear reasonable, it is widely recognized that there are significant uncertainties in the prediction of adverse seepage conditions, uplift pressures, internal erosion, or piping conditions over time. Inspection and emergency action plans for a dam are therefore important components of guarding against potential consequences of such conditions. These aspects were, however, outside the scope of our review.

3. Maximum Design Earthquake Ground Motions and Time Histories:

The evaluation of seismic sources, the development of the maximum design earthquake (MDE) ground motion spectra, and the selection of ground motion time series are generally reasonable. The use of 84^{th} percentile motions for the MDE event could be considered overly conservative because the Foothills Fault System has a very low slip rate (0.05 mm/year) and the 84^{th} percentile motions have an annual return period that is greater than about 20,000 years. However, the results of sensitivity studies in the T/GP JV team's evaluation of seismic deformations, as discussed later, suggest that the final assessment of seismic performance would likely be comparable even if the 50^{th} percentile motions had been used.

The inclusion of a San Andreas event and the corresponding ground motion time series are also reasonable. The results of the analyses for the San Andreas event provided valuable insight on the likely performance of the dam during more frequent events.

The three selected time series for the MDE appear reasonable and sufficient. It is often necessary to use more than three time series in seismic deformation analyses, but the results of the sensitivity analyses in the T/GP JV team's evaluation of seismic deformations, as discussed later, suggest that the deformations will not be significantly affected by reasonable variations in the time series' characteristics.

The Travasarou et al (2002) relationships for estimating Arias Intensity hazard predated completion of the NGA database and exhibit trends at near source distances that are inconsistent with current understanding of ground motion attenuation. We suggest that this relationship not be used in checking the adequacy of ground motion time series. Because of the improved site categorization used in the NGA research project, we recommend that you consider using only the relationship derived by Watson-Lamprey and Abrahamson (2006) for estimating the target Arias Intensity because it was based on the NGA data.

The fault normal factors in two of the five NGA relationships overestimate the fault normal effect in our opinion. It appears that the T/GP JV team did not include the fault normal factors in the development of the MDE design motions, although this detail requires clarification, for exactly this reason. We would concur with the omission of the fault normal factors.

4. Seismic Deformation of Camanche Reservoir Main Dam:

The approach and procedures used for the seismic deformation analyses of the main dam were reasonable. The T/GP JV team performed two-dimensional equivalent-linear dynamic analyses using QUAD4M and two-dimensional nonlinear static and dynamic analyses using FLAC. The dynamic responses in the absence of liquefaction were in good agreement for both models. FLAC analyses were used to assess the effects of liquefaction on dynamic deformations and on post-seismic (gravity driven) deformations. Limit equilibrium slope stability analyses were used to check the consistency between post-seismic static factors of safety and the results of the post-seismic deformation analyses using FLAC. Deformations were computed for the three MDE time series and the one San Andreas event time series. Sensitivity analyses were performed to evaluate the effect of varying the residual strength of liquefied tailings and other input parameters. The comparisons of the QUAD4M and FLAC analyses, the systematic evaluation of whether deformations are driven by gravity or by inertia, and the various sensitivity analyses provided valuable insights on the mechanisms and parameters governing the computed deformations.

The dynamic analyses for the MDE event predict that liquefaction will occur in the tailings at the upstream and downstream toes and beneath the shells for short distances away from the toes. The fact that liquefaction does not extend to large distances beneath the shells was shown to be a consequence of two primary factors: (1) the cyclic stress ratios or CSR decreasing with distance from the toe, and (2) the cyclic resistance ratios or CRR increasing with distance from the toe because the $(N_1)_{60cs}$ values are increasing (Figures 6 and 7 in the T/GP JV report). The results of sensitivity analyses and the relative magnitudes of CSR and CRR values indicate that this conclusion is relatively insensitive to reasonable variations in the material characterization or the MDE design motions (e.g., 50th percentile vs. 84th percentile motions).

Liquefaction of the tailings near the toes of the main dam, where the $(N_1)_{60cs}$ values are lowest, appears likely for the MDE motions and possibly also for the more frequent San Andreas event. The prediction of liquefaction near the toes during these events appears to be relatively insensitive to reasonable variations in the material characterization or ground motion characteristics. This finding appears reasonable in view of the low $(N_1)_{60cs}$ values, the high ground water levels, and the design ground motions.

The dynamic deformation analyses conservatively assumed that liquefaction would be triggered throughout the tailings at the beginning of seismic shaking. The computed deformations were mostly concentrated around the upstream and downstream toes of the dam, with deformations near the crest being significantly smaller. This pattern of deformations is consistent with the fact that the tailings materials beneath the shells were assigned residual shear strengths that increased significantly with distance from the toes (i.e., because the $(N_1)_{60cs}$ values increased with distance from the toes). The deformations near the toes were shown to be dominated by gravity effects and not by inertia effects; Figure 12 shows that once liquefaction is triggered, inertia contributes little to the final deformation. For this reason, the use of only three MDE ground motion time series seems reasonable, as the dispersion of predicted deformations due to different ground motions is not likely to be significant.

In some situations, deformations at the toe of an embankment can lead to a progression of retrogressive slides up toward the crest. We agree that a qualitative evaluation of the characteristics of the main dam suggest that the potential impacts of any retrogressive sliding are unlikely to be significant. For example, the well-compacted shell materials can likely stand at slope angles significantly steeper that those of the current upstream and downstream faces, such that the loss of material from the toe areas in unlikely to cause a failure mechanism that could seriously impact the crest. The issue of retrogressive sliding, however, and the reasons why it is unlikely for the main dam should be addressed in the final report.

The computed crest displacements for the 84th percentile MDE motions, even with the conservative assumption that all the tailings liquefy early in shaking, were only about one foot. This magnitude of crest displacement appears reasonable given the height of the dam, the well-compacted nature of the core and shells, and the estimated distribution of $(N_1)_{60cs}$ values in the tailings beneath the shells. It appears that a more conservative interpretation of the $(N_1)_{60cs}$ values in the tailings beneath the shells would not significantly affect this conclusion if the extent and timing of liquefaction triggering was accounted for less conservatively.

The seismic deformation analysis results indicate that the main dam would be expected to perform acceptably well during the MDE, subject to the expectation that the further examination of the relief well system shows that liquefaction-induced damage to the relief wells would not impair the safe performance of the dam. The computed deformations are relatively small, while the dam is capable of withstanding significantly large deformations given that it has 27.5 feet of freeboard, a wide core, and transition zones that are reported to meet modern filter criteria.

We suggested that the consequence of liquefaction-induced damage to the relief wells of the main dam be evaluated, including examining the alternative choices of actions, in sufficient detail to provide an informed basis for decision making.

- The analyses of liquefaction triggering and deformations could be extended to evaluate the likelihood of various amounts of ground deformation near the relief wells for the case of no remediation.
- The potential consequences of damage to the relief wells (e.g., how would the relief wells be repaired and what would the cost be?) and to reservoir operations (e.g., would the reservoir need to be drawn down, and at what cost?) could be evaluated.
- Alternative remediation options could be evaluated for conceptual feasibility and likely costs. For example, it appears that several ground improvement methods (e.g., dynamic compaction, stone columns, and deep soil mixing) would be feasible because the site access is good and the problem soils are relatively shallow.

The results of the above studies would provide a means for evaluating the relative merits of remediating the site now versus accepting the risks and fixing any damages that occur near the toes in future earthquakes.

<u>Closure</u>

We hope that the T/GP JV team finds these comments helpful as they continue their work on the Camanche Embankments Safety Review Project. If you have any questions or require clarification on any of the comments contained herein, please feel free to contact either of us.

Respectfully submitted,

Ross W. Boulanger

I. M. Idriss

Attachment A: Meeting Attendance Lists
Attachment B: Meeting Agenda

Attachment A: Meeting Attendance Lists

Meridien Hotel on June 29th, 2009

I. M. Idriss	ITRB
R. W. Boulanger	ITRB
Patrick Allen	T/GP JV
John Barneich	T/GP JV
Bob Kirby	T/GP JV
Yoshi Moriwaki	T/GP JV
Guilane Roussel	T/GP JV
Dar Chen	EBM
Atta Yiadom	EBM

East Bay Municipal Utilities District Office on June 30, 2009

I. M. Idriss	ITRB
R. W. Boulanger	ITRB
John Barneich	T/GP JV
Bob Kirby	T/GP JV
Yoshi Moriwaki	T/GP JV
Guilane Roussel	T/GP JV
Elizabeth Bialek	EBMUD
Dar Chen	EBMUD
Atta Yiadom	EBMUD
Yogesh Prashar	EBMUD
Humphrey Chan	EBMUD

TERRA / GeoPentech
a Joint Venture

Meeting Agenda

CAMANCHE EMBANKMENTS SAFETY REVIEW

INTERNAL TECHNICAL REVIEW BOARD MEETING
June 29 and 30, 2009

Purpose of Meeting

The results of the analyses to date suggest that the MDE, and to a lesser extent an event on the San Andreas Fault, are likely to trigger liquefaction in the dam foundation and generate enough deformation to cause the failure of the relief wells along the toe of the dam. These results are a surprise to the East Bay Municipal Utility District (District) because they differ from what the District had been led to believe by previous analyses. T/GP needs to critically evaluate our work to check that our analyses do not include unnecessary conservatism.

The main purpose of the meeting is to brief our Internal Technical Review Board (ITRB) on the approach and assumptions used in the analyses and to obtain the ITRB's candid review and comments on these analyses, looking for opportunities to reduce conservatism, as appropriate.

Agenda

June 29, 2009

10:00 AM – 12:00 PM	Introduction Summary of Analyses • Overview of Previous Analyses by Wahler • Site Characterization • Seepage Analyses with and without Relief Wells • Ground Motions • Seismic Deformation of Dam
12:00 PM – 1:00 PM	Lunch
1:00 PM - 5:30 PM	Discussion and ITRB Comments

June 30, 2009

9:00 AM – 11:00 AM	Briefing of District on Results of Analyses and ITRB Review Comments
11:00 AM – 2:00 PM	ITRB writes short letter-report (lunch will be brought in)
2:00 PM – 5:00 PM	Field Trip to San Pablo Dam