## PMComanchePeakPEm Resource

**From:** Monarque, Stephen
**Sent:** Monday, November 02, 2009 5:00 PM
**To:** ComanchePeakCOL Resource
**Subject:** FW: 2009-10-22 Presentations made during geology site visit
**Attachments:** ShearVel.pdf; 2-5-4_Overview.pdf; 2-5-4_RAIs.pdf; CP_SiteAudit_CrinerMEEG.pdf

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**From:** Donald.Woodlan@luminant.com [mailto:Donald.Woodlan@luminant.com]
**Sent:** Thursday, October 22, 2009 1:14 PM
**To:** Monarque, Stephen
**Cc:** John.Conly@luminant.com
**Subject:** 2009-10-22 Presentations made during geology site visit

The four presentations that you have attached are the only presentations that we made during the site visit. Per my memory (and my notes), the only presentation used or discussed during the public meetings is the “2-5-4 RAIs.pdf” presentation.

*D. R. Woodlan*
Donald R. Woodlan
**Luminant**
bus 254-897-6887, cell 214-542-7761, fax 254-897-6890
Donald.Woodlan@luminant.com

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**From:** Monarque, Stephen [mailto:Stephen.Monarque@nrc.gov]
**Sent:** Tuesday, October 20, 2009 2:44 PM
**To:** Woodlan, Don; Conly, John
**Subject:** FW: 2009-07-29 Presentations made during geology site visit

Don/John,

Other than the 2.5.4 RAI presentation, was there any other presentation that was discussed during the public meetings?

thanks,
Steve

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**From:** Donald.Woodlan@luminant.com [mailto:Donald.Woodlan@luminant.com]
**Sent:** Thursday, July 30, 2009 2:15 PM
**To:** Kallan, Paul
**Cc:** Monarque, Stephen
**Subject:** 2009-07-29 Presentations made during geology site visit

See attached.

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**Hearing Identifier:** ComanchePeak_COL_Public
**Email Number:** 714

**Mail Envelope Properties** (9C2386A0C0BC584684916F7A0482B6CA05889DDC77)

**Subject:** FW: 2009-10-22 Presentations made during geology site visit
**Sent Date:** 11/2/2009 5:00:13 PM
**Received Date:** 11/2/2009 5:00:15 PM
**From:** Monarque, Stephen

**Created By:** Stephen.Monarque@nrc.gov

**Recipients:**
"ComanchePeakCOL Resource" <ComanchePeakCOL.Resource@nrc.gov>
Tracking Status: None

**Post Office:** HQCLSTR02.nrc.gov

| **Files** | **Size** | **Date & Time** |
|---|---|---|
| MESSAGE | 2562 | 11/2/2009 5:00:15 PM |
| ShearVel.pdf | 504127 | |
| 2-5-4_Overview.pdf | 804958 | |
| 2-5-4_RAIs.pdf | 107107 | |
| CP_SiteAudit_CrinerMEEG.pdf | 1887322 | |

**Options**
**Priority:** Standard
**Return Notification:** No
**Reply Requested:** No
**Sensitivity:** Normal
**Expiration Date:**
**Recipients Received:**

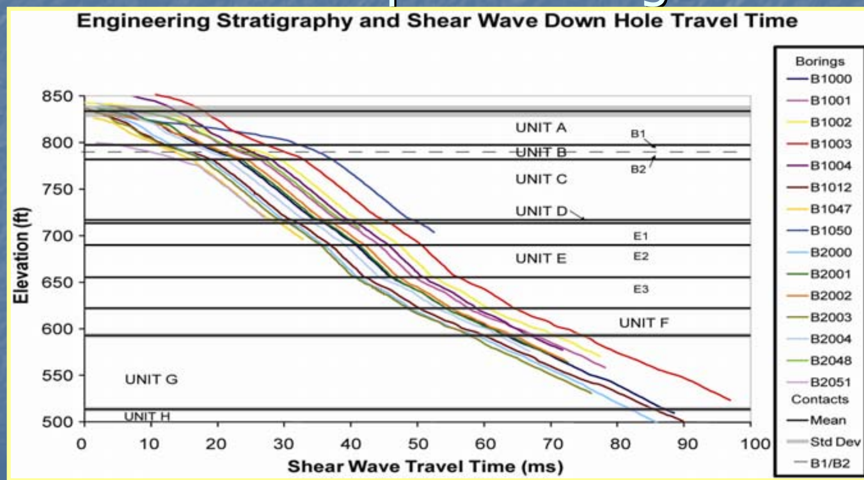
# Overview

- Quick review velocity information and site engineering stratigraphy
- Development of shallow profile (<550 ft)
- Comparison of results from different geophysical methods (Suspension Log – SASW – Downhole – Crosshole)
- Development of deep profile to "seismic basement" (i.e. Vs=9200 ft/sec: 550 - ~5500 ft)

# Suspension Log, SASW and Downhole Velocity Control


- 15 borings with suspension logs
- 4 borings with co-located SASW inversions
- 2 borings with co-located downhole profiles
- Unit 1 & 2 crosshole

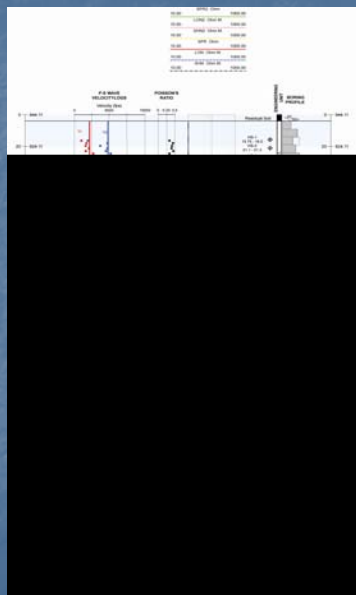
# Engineering Stratigraphy and Simulated Downhole Travel Times from Suspension Logs




# Shallow Profile Development Methodology

- Parse SSL simulated downhole travel times by layer
- Linear regression on times vs depth – slope gives the layer slowness (invert slowness to get representative $V_s$ and $V_p$ for each layer).
- Use representative Vs and Vp for each layer to calculate a representative Poisson's Ratio – these values with the engineering layer boundaries constitute boring specific profiles (15)

# B1012 Boring Summary Log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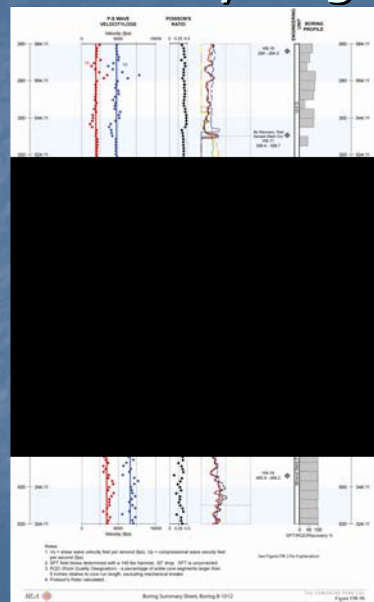
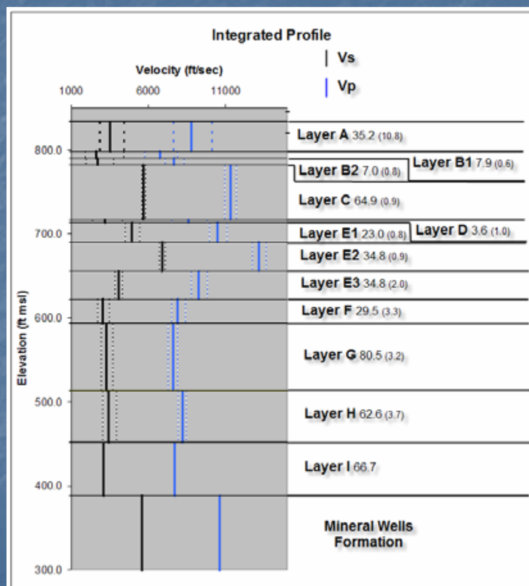

[redacted]

# Develop Integrated Site Profile

- Combine representative layer Vs an Vp for each boring to produce an integrated Vs and Vp for each layer (take the geometric mean and log deviates)
- Calculate Poisson's Ratio for each layer using the integrated Vs and Vp values for each layer
- Combine the results from above with the engineering stratigraphic boundaries (+/- sd) and thicknesses (+/- sd) to produce integrated profile



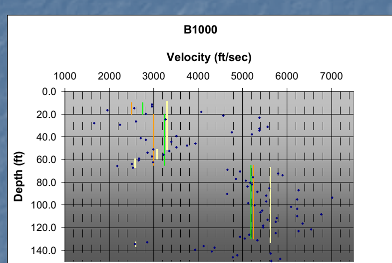

# Layer C Representative Velocities

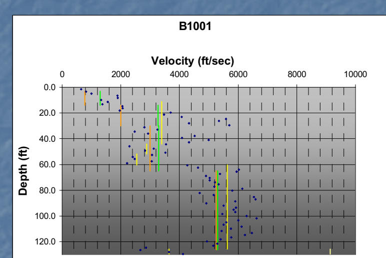
| Layer C | | | | |
|---|---|---|---|---|
| **Boring** | **Vs** | **Log10(Vs)** | **Vp** | **Log10(Vp)** |
| **B1000** | 5585 | 3.7470 | 11177 | 4.0483 |
| **B1001** | 5613 | 3.7492 | 11194 | 4.0490 |
| **B1002** | 5667 | 3.7534 | 11321 | 4.0539 |
| **B1003** | 5623 | 3.7500 | 11063 | 4.0439 |
| **B1004** | 5704 | 3.7562 | 11161 | 4.0477 |
| **B1012** | 5660 | 3.7528 | 10928 | 4.0385 |
| **B1047** | 5625 | 3.7501 | 11557 | 4.0628 |
| **B1050** | 5598 | 3.7480 | 11493 | 4.0604 |
| **B2000** | 5738 | 3.7588 | 11375 | 4.0560 |
| **B2001** | 5707 | 3.7564 | 11388 | 4.0564 |
| **B2002** | 5731 | 3.7582 | 11185 | 4.0486 |
| **B2003** | 5758 | 3.7603 | 11305 | 4.0533 |
| **B2004** | 5643 | 3.7515 | 11372 | 4.0558 |
| **B2048** | 5648 | 3.7519 | 11609 | 4.0648 |
| **B2051** | 5539 | 3.7434 | 11336 | 4.0545 |
| **Mean Logs:** | 3.7525 | | 4.0529 | |
| **Geometric Mean:** | **5656** | | **11296** | |
| **Arith. Mean:** | **5656** | | **11298** | |
| **Std:** | **62** | | **184** | |

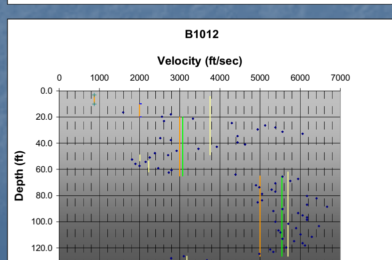
# SASW Velocity Control

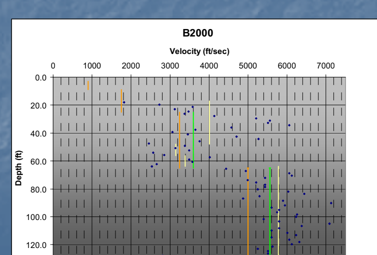

- 4 borings with co-located SASW inversions

# R1-R2 Suspension Vs – SASW Vs Comparison

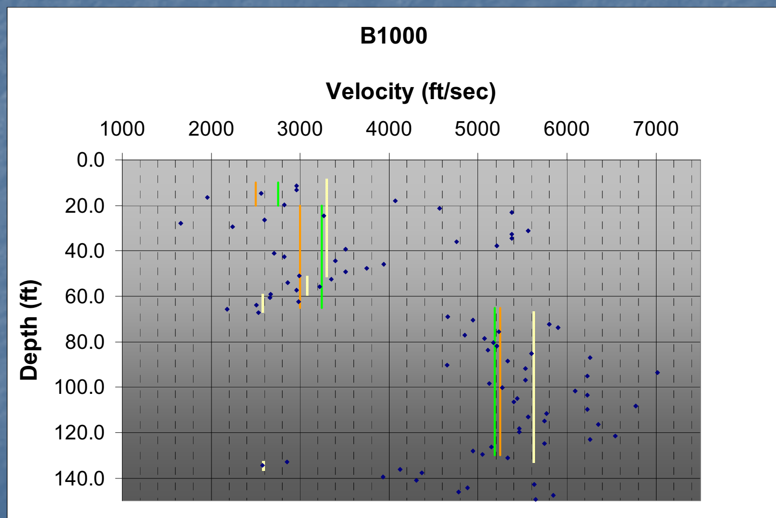


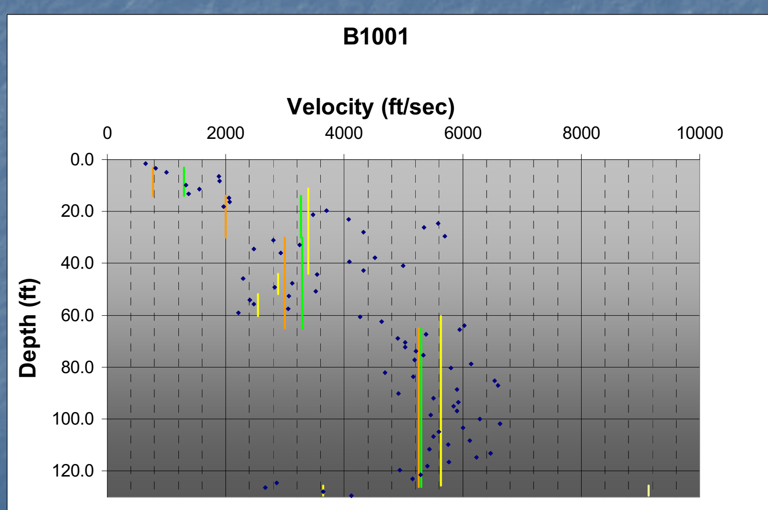



## SASW



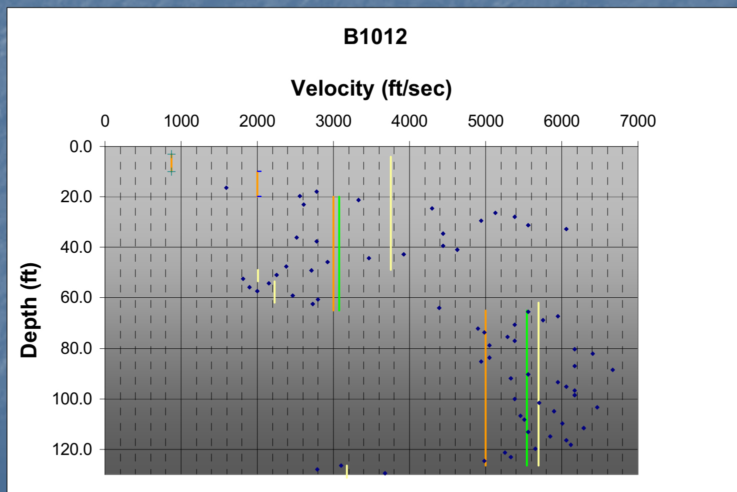
B1000
Velocity (ft/sec)
1000
2000
3000
4000
5000
6000
7000
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
Depth (ft)


## SASW

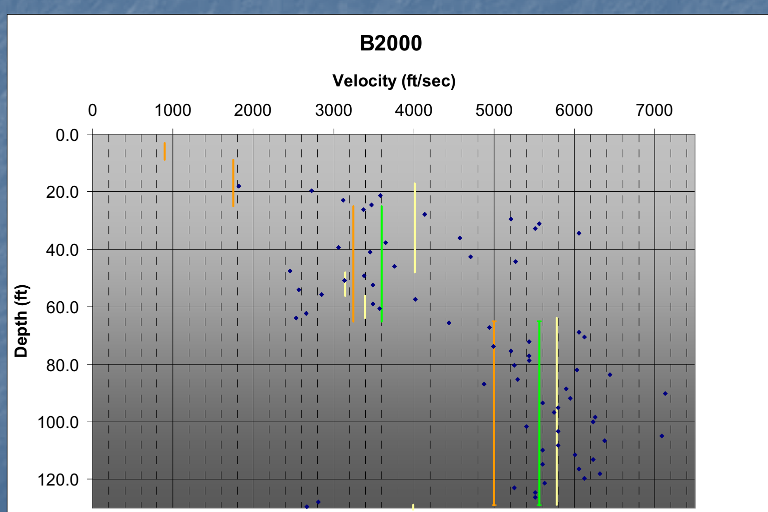


B1001
Velocity (ft/sec)
0
2000
4000
6000
8000
10000
0.0
20.0
40.0
60.0
80.0
100.0
120.0
Depth (ft)

## SASW


B1012
Velocity (ft/sec)
0
1000
2000
3000
4000
5000
6000
7000
0.0
20.0
40.0
60.0
80.0
100.0
120.0
Depth (ft)

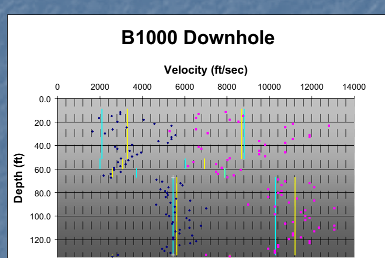
## SASW

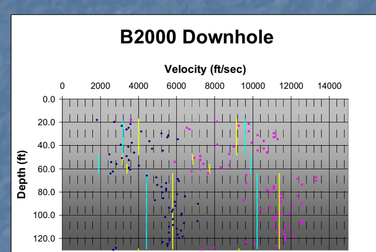

B2000
Velocity (ft/sec)
0
1000
2000
3000
4000
5000
6000
7000
0.0
20.0
40.0
60.0
80.0
100.0
120.0
Depth (ft)

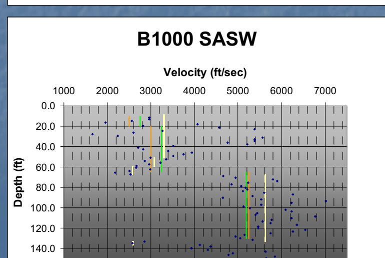
# Downhole Velocity Control

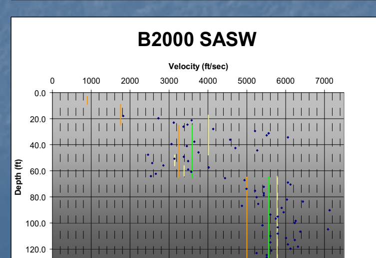

- 2 borings with co-located downhole profiles

# R1 – R2 Suspension Velocity – Downhole Comparison

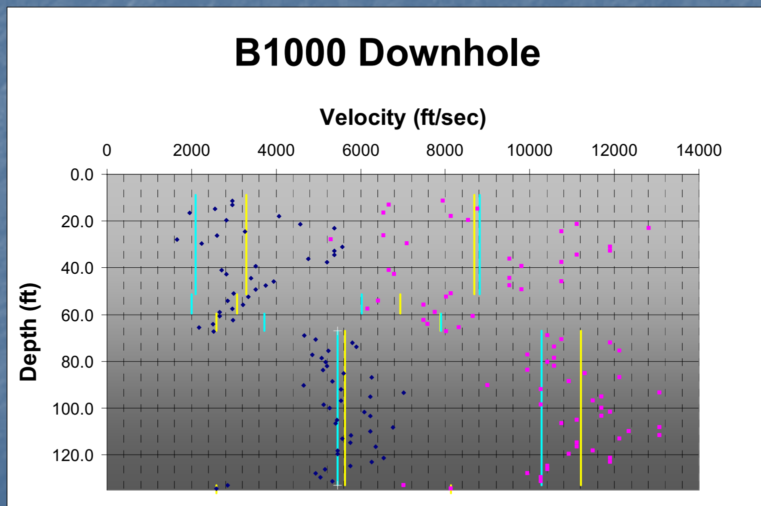


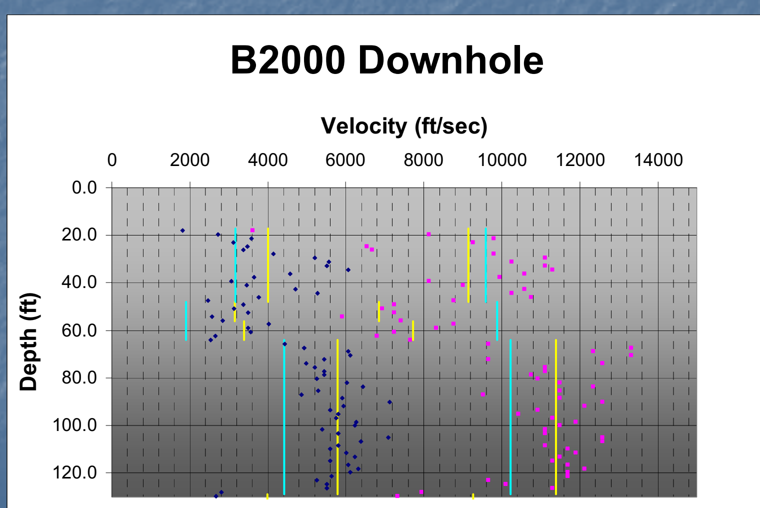



# Downhole

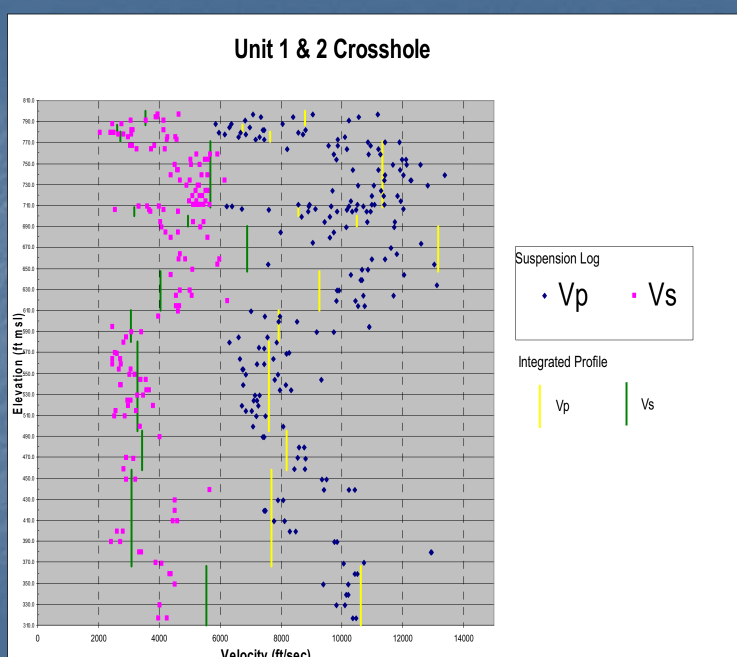



# Downhole

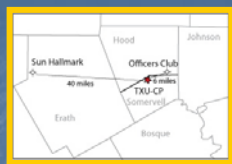


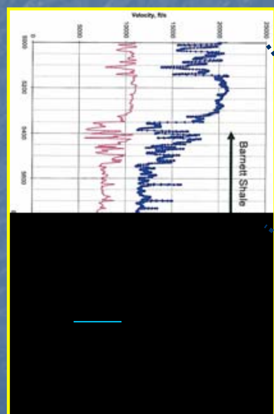
# Suspension Log, SASW and Downhole Velocity Control


- Unit 1 & 2 crosshole

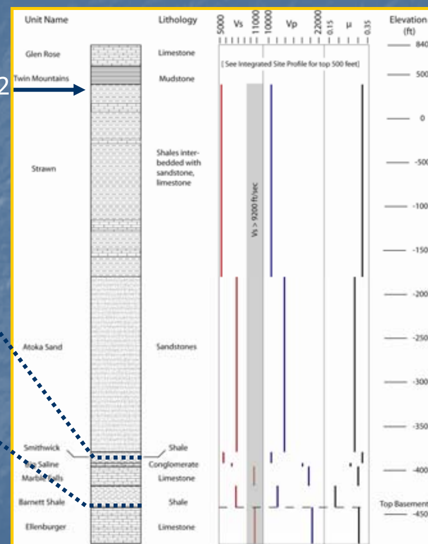


# Deep Profile




B1012

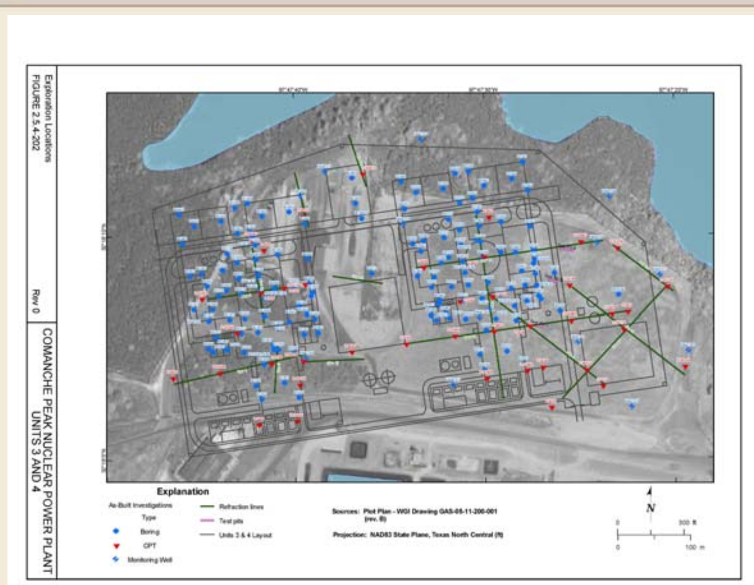


# Comanche Peak Units 3 & 4

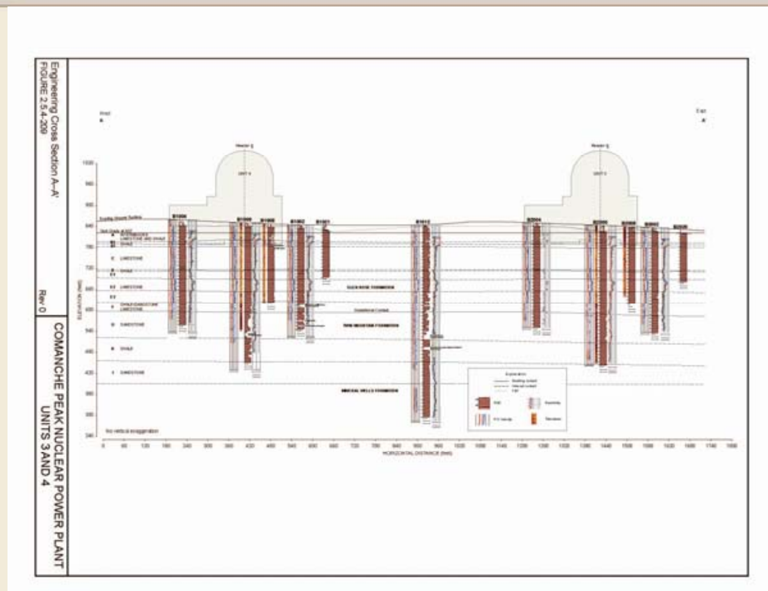
NRC Geology Safety Site Visit – July 28-30, 2009
Geotechnical Engineering Overview





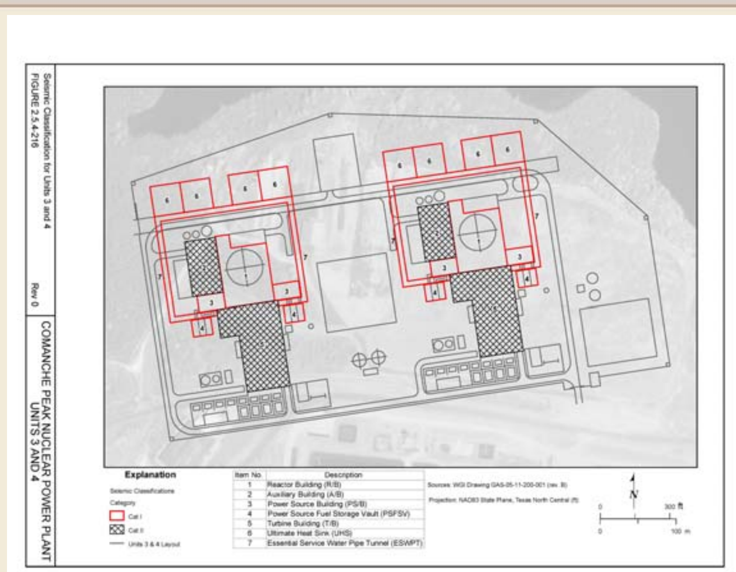


## Comanche Peak Units 3 & 4






## Comanche Peak Units 3 & 4

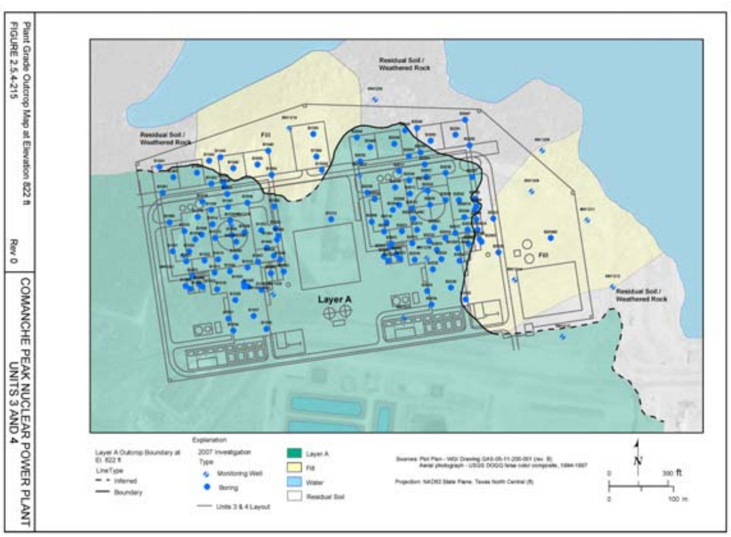


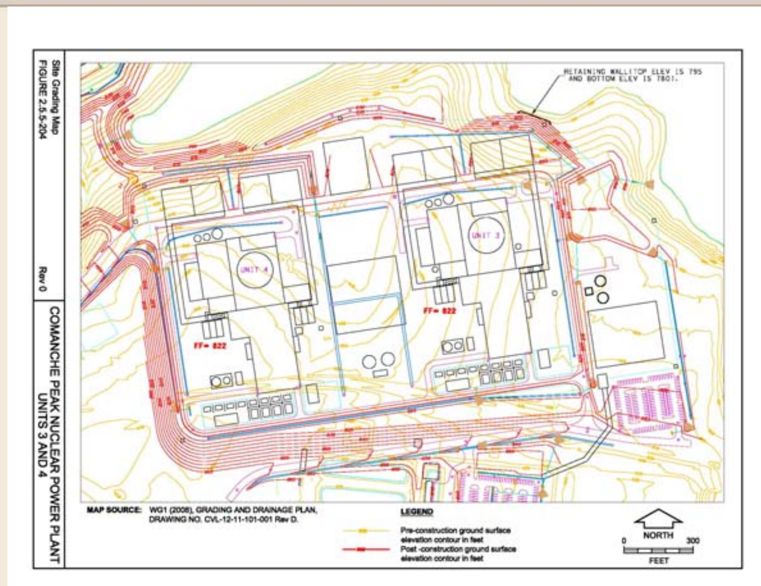
### Main Seismic Category I & II Structures' Details

| Building | Category | Foundation Length (ft) | | Foundation Bottom Elev. (ft) | Fill Concrete Thickness below Foundations (ft) | Subgrade Below Foundations |
|---|---|---|---|---|---|---|
| | | E-W | N-S | | | |
| R/B | I | 213'-4" | 308'-11" | 783'-2" | ~ 1'-2" | Fill Concrete over Limestone |
| T/B | II | 185'-10" | 314'-10" | 794'-10" | ~ 12'-10" | Fill Concrete over Limestone |
| A/B | II | 133'-4" | 239'-3" | 784'-9" | ~ 2'-9" | Fill Concrete over Limestone |
| EPS/B | I | 114'-10" | 69'-4" | 784'-9" | ~ 2'-9" | Fill Concrete over Limestone |
| WPS/B | I | 114'-10" | 69'-4" | 784'-9" | ~ 2'-9" | Fill Concrete over Limestone |
| PSFSV | I | 85' | 78' | 782' | ~ 0' | Fill Concrete over Limestone |
| UHS | I | 131' | 131' | 775' to 787' | ~ 0' to 5' | Fill Concrete over Limestone |
| ESWPT | I | 25' (Tunnel Width) | | 791' | ~ 9' | Fill Concrete over Limestone |
| Duct Banks | I | 3' to 6' (Duct Width) | | 818' to 819' | NA | Engineered Fill over Limestone or Fill Concrete |














## Site Grading

- Finish Floor Elevation = 822 ft
- Unit 3 Power Block Existing Grade = 830 ft – 855 ft (Cuts of 8-33 ft)
- Unit 4 Power Block Existing Grade = 842 ft – 865 ft (Cuts of 20-46 ft)

- Observations and monitoring are required to be performed during 1) general excavation, to achieve mat foundation bearing elevations, 2) additional excavations below the design mat bearing elevations, and 3) cleanout of any defects in the rock foundation. The exposed excavation bottoms also need to be mapped by the project engineering geologist.
- Geologic mapping of final exposed rock surfaces beneath Units 3 and 4, and any required extension to reach suitable rock material, is periodically carried out at a scale of 1 in equals 5 ft. Areas where further detail is needed for documentation of significant features are also documented on the geologic map.
- The geologic mapping program includes photographic documentation of exposed surfaces and laboratory testing and documentation of significant features.
- Similar to Units 1 and 2 foundation excavations, extensometers are also needed during foundation excavation for Units 3 and 4 to monitor foundation deformation.

## Laboratory Testing (Project Report: TXUT-001-PR-010)

**Soil Samples:**
- Grain-Size Distribution (11)
- Grain-Size Distribution with Hydrometer (7)
- Moisture Content (8)
- Atterberg Limits (17)
- Organic Content (2)

**Rock Samples:**
- Moisture Content and Unit Weight (206)
- Atterberg Limits (33)
- Calcium Carbonate (28)
- Specific Gravity (13)
- Slake Durability (7)
- One-Dimensional Consolidation (5)
- One-Dimensional Swell (9)
- Unconfined Compression (64)
- Point Load Index (43)
- Consolidated-Undrained Triaxial (40)
- Unconsolidated-Undrained Triaxial (11)
- Direct Shear (7)
- Shear Wave Velocity (81)
- Petrographic Analysis (39)
- X-Ray Diffraction Analysis (14)





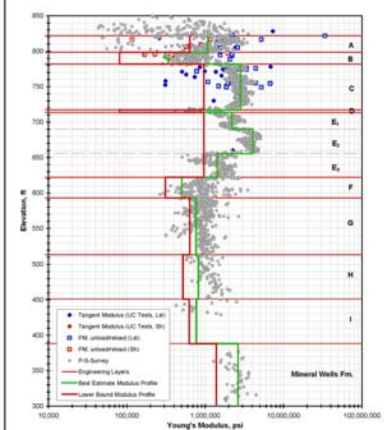
## Settlement Analysis

Rock Deformation Model

- Rock Mass Modulus “Best Estimate”
  - Shear Wave Velocity Data
  - Local influence of Discontinuities & Variations
  - Adjusted for Strain Level

- Rock Mass Modulus “Lower Bound”
  - Laboratory Test Data
  - RMR & GSI System (Hoek et al.) for Discontinuities & Variations
  - Empirical Relationships (Nicholson et al., Mitri et al., Sonmez et al., & Hoek et al.)

## Comanche Peak Units 3 & 4




Elastic Modulus Models for Settlement Analysis
FIGURE 2.5.4-241 Rev 0

COMANCHE PEAK NUCLEAR POWER PLANT
UNITS 3 AND 4



## Comanche Peak Units 3 & 4

**Comanche Peak Nuclear Power Plant, Units 3 & 4**
**COL Application**
**Part 2, FSAR**

**TABLE 2.5.4-229**
**SUMMARY OF SETTLEMENT ESTIMATES BASED ON "BE" PROFILE**

| Structure | Category | Foundation Size (ft) | | Foundation Bottom Elev. (ft) | Foundation Static Load (ksf) | Settlement Estimate for Center (in) | |
|---|---|---|---|---|---|---|---|
| | | E-W | N-S | | | Non-Layered Method | Layered Method |
| R/B | I | 213 | 309 | 783 | 11.3 | 0.12 | 0.20 |
| T/B | II | 186 | 315 | 795 | 5.9 | 0.07 | 0.11 |
| A/B | II | 133 | 239 | 785 | 6.8 | 0.09 | 0.14 |
| EPS/B | I | 115 | 69 | 785 | 4.3 | 0.07 | 0.10 |
| WPS/B | I | 115 | 69 | 785 | 4.3 | 0.08 | 0.12 |
| PSFSV | I | 85 | 78 | 782 | 5.4 | 0.06 | 0.09 |
| UHS | I | 131 | 131 | 787 | 3.6 | 0.05 | 0.06 |

Comanche Peak Nuclear Power Plant, Units 3 & 4
COL Application
Part 2, FSAR

TABLE 2.5.4-230

SUMMARY OF SETTLEMENT ESTIMATES BASED ON "LB" PROFILE

| Structure | Category | Foundation Size (ft) | | Foundation Bottom Elev. (ft) | Foundation Static Load (ksf) | Settlement Estimate for Center (in) | |
|---|---|---|---|---|---|---|---|
| | | E-W | N-S | | | Non-Layered Method | Layered Method |
| R/B | I | 213 | 309 | 783 | 11.3 | 0.30 | 0.37 |
| T/B | II | 186 | 315 | 795 | 5.9 | 0.19 | 0.20 |
| A/B | II | 133 | 239 | 785 | 6.8 | 0.23 | 0.26 |
| EPS/B | I | 115 | 69 | 785 | 4.3 | 0.18 | 0.18 |
| WPS/B | I | 115 | 69 | 785 | 4.3 | 0.20 | 0.21 |
| PSFSV | I | 85 | 78 | 782 | 5.4 | 0.17 | 0.16 |
| UHS | I | 131 | 131 | 787 | 3.6 | 0.14 | 0.12 |





## Bearing Capacity

- **General Shear Failure**
- **Local Shear Failure**
- **Compressive Failure**

Comanche Peak Nuclear Power Plant, Units 3 & 4
COL Application
Part 2, FSAR

TABLE 2.5.4-228

SUMMARY OF ULTIMATE BEARING CAPACITIES

| Structure | Category | Foundation Size (ft) | | Foundation Bottom Elev. (ft) | Ultimate Bearing Capacity (ksf) | | |
|---|---|---|---|---|---|---|---|
| | | E-W | N-S | | General Shear | Local Shear | Compression |
| R/B | I | 213 | 309 | 783 | 354 | 348 | 146 |
| T/B | II | 186 | 315 | 795 | 342 | 339 | 146 |
| A/B | II | 133 | 239 | 785 | 338 | 335 | 146 |
| EPS/B | I | 115 | 69 | 785 | 343 | 340 | 146 |
| WPS/B | I | 115 | 69 | 785 | 343 | 340 | 146 |
| PSFSV | I | 85 | 78 | 782 | 365 | 362 | 146 |
| UHS | I | 131 | 131 | 787 | 369 | 365 | 146 |

Comanche Peak Nuclear Power Plant, Units 3 & 4
COL Application
Part 2, FSAR

Table 3.8-202
Summary of Bearing Pressures and Factor of Safety

| Building | Bearing Pressures (ksf) | | Ultimate Bearing Capacity (ksf) | Available Factor of Safety | |
|---|---|---|---|---|---|
| | Static Case | Seismic Case(1),(2) | | Static Case | Seismic Case |
| R/B | 11.3 | 18.9 | 146.00 | 12.9 | 7.7 |
| T/B | 5.9 | 7.4 | 146.00 | 24.7 | 19.7 |
| A/B | 6.6 | 10.8 | 146.00 | 22.1 | 13.5 |
| PS/Bs | 4.3 | 7.4 | 146.00 | 34 | 19.7 |
| PSFSVs | 2.9(3) | 5.1(3) | 146.00 | 50.3 | 28.6 |
| UHSRS | 4.5(4) | 16.2(4) | 146.00 | 32.4 | 9.0 |
| ESWPT | 3.6(5) | 12.4(5) | 146.00 | 40.6 | 11.8 |

Notes:

1) All seismic case bearing pressures are based on the site-specific FIRS with 0.1 g PGA as described in Subsection 3.7.1.
2) Seismic case bearing pressures shown above include static bearing pressures.
3) The pressure shown includes bearing pressure due to full fuel oil tanks.
4) The pressure shown includes bearing pressure due to full reservoirs.
5) The maximum bearing pressures occur underneath the portion of the ESWPT supporting the air intake missile shields adjacent to the UHSRS.

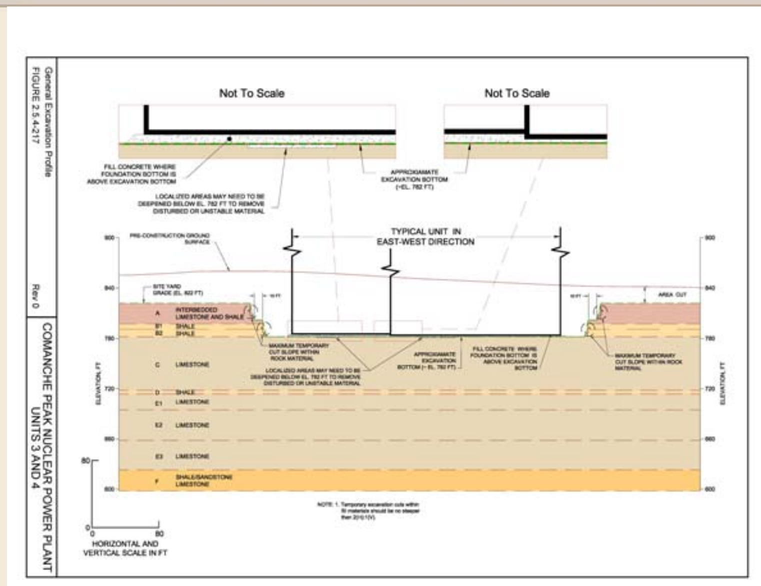




**Lateral Earth Pressures** (CALC. No.: TXUT-001-FSAR-2.5-CALC-010)

- Lateral Static Pressures
  - Backfill Pressures
    - Yielding Walls (Level & Sloping Backfill)
    - Unyielding Walls
  - Surcharge Pressures
  - Hydrostatic Pressures
  - Compaction Pressures
  - Lateral Seismic Loads
  - Rock Lateral Pressures
  - Resistance to Lateral Loads
    - Friction Resistance
    - Passive Resistance

## Comanche Peak Units 3 & 4






## Comanche Peak Units 3 & 4

**Structural Fill Specification**

- Structural Fill is used in excavated areas around Units 3 & 4 and north-facing fill slope areas adjacent to SCR.
- Should consist of durable materials free from organic matters or any other deleterious or perishable substances, and shall be of such nature that it can be compacted readily by watering and rolling to a firm and non-yielding state,
- Should be granular in nature, with a well-graded grain size distribution and less than 30 percent by weight passing standard US Sieve No. 200 (ASTM D422 and D1140),
- Should not contain particles greater than 3 inches in the maximum dimension, with less than 15 percent by weight larger than 2.5 inches,
- Should have an expansion index (ASTM D4829) less than 20; material otherwise deemed to be expansive and is not acceptable,
- Should have a liquid limit less than 40 percent, and a plasticity index not exceeding 12 (ASTM D4318)., and
- Should be placed in lifts no thicker than 8 inches (measured in loose state); moisture conditioned to at least within 2 percent of the optimum moisture content and compacted to a minimum relative compaction of 95 percent (ASTM D1557).

Fill is derived from either off-site borrow areas or on-site cut areas and foundation excavations. The excavated materials from on-site areas require appropriate segregation, handling, and processing. Geotechnical testing is required for all fill materials to verify that their characteristics and properties meet the minimum requirements.

**Compacted Fill Soil Design Parameters**

- Based on the specification requirements, the geotechnical properties for the compacted granular fill materials are estimated as follow:
- Total Unit Weight (s) ~ 125 pcf
- Internal Friction Angle ($\phi$) ~ 32°
- Shear Wave Velocity (Vs):
  - 650 fps for 0 to 3 ft depth (Variation: 325 – 975),
  - 800 fps for 3 to 20 ft depth (Variation: 400 – 1200), and
  - 1000 fps for 20-50 ft depth (Variation: 500 – 1500).
- Poisson's Ratio (ave.) ~ 0.35

Verification seismic velocity and resonant column/torsional shear (RCTS) testing on compacted fill material placed around seismic Category I and II structures are required to confirm that the non-linear properties of the fill are within the above described variability.



Comanche Peak Units 3 & 4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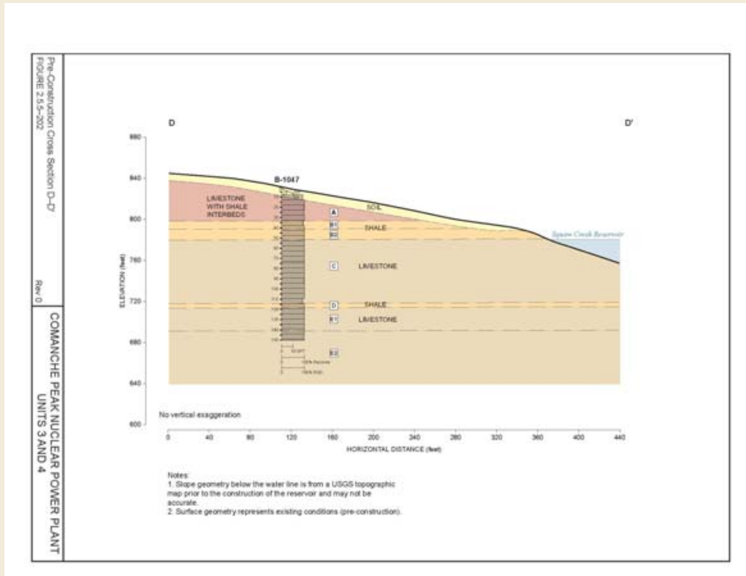
## Comanche Peak Units 3 & 4

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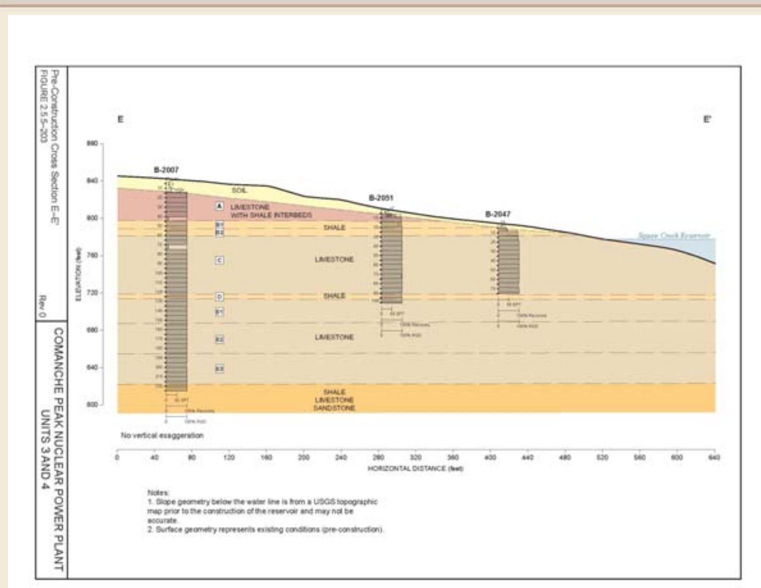
## Comanche Peak Units 3 & 4

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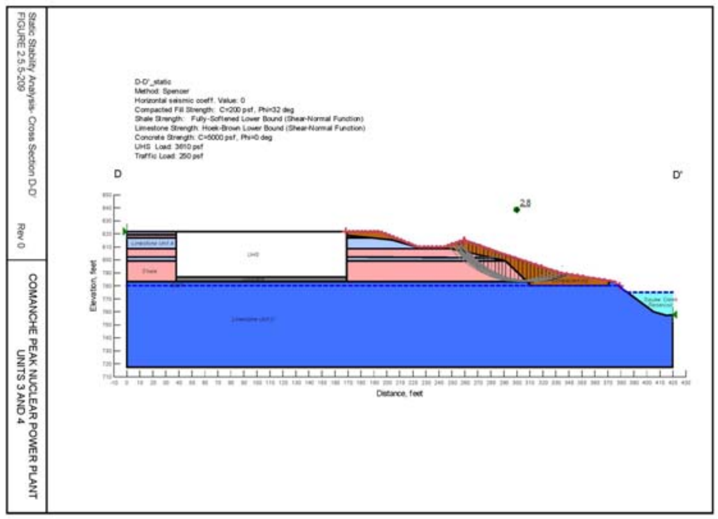
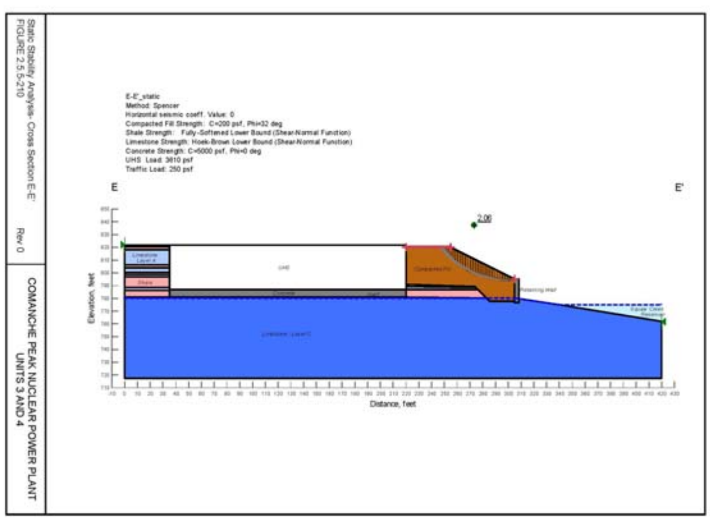
## Comanche Peak Units 3 & 4






## Comanche Peak Units 3 & 4



# Comanche Peak Units 3 & 4

NRC Geology Safety Site Visit – July 28-30, 2009
RAIs 19 and 22





## RAI 2.5.4-1

**The Comanche Peak Units 3 and 4 (CPNPP) Final Safety Analysis Report (FSAR), in Section 2.5.4 (for example pages 121 and 129), states that the site "conforms to a relatively uniform site condition." The laboratory data obtained for samples tested from immediately beneath and to the sides of the power block structures, indicates potentially significant variability in properties (see, for example, Figures 2.5.4-219 and onward, data ranges described in Sections 2.5.4.2.3.1.1 and 2.5.4.2.3.3). Please provide the criteria used to make the judgment that the proposed site "conforms to a relatively uniform site condition[,]" and indicate if the assessment is appropriate for both site response and soil-structure interaction (SSI) assessments for which specific uniformity criteria are assumed.**

# RAI 2.5.4-2

**Section 2.5.4.1.2 (page 124) of the FSAR indicates that the materials beneath the footprint of the facilities may contain localized zones or thin beds of poorly cemented or soft materials. These materials are discounted from having an important effect on response of performance or stability of the plant foundations on the basis of the small percentage of thickness of these materials as compared to the total thickness of the layer. Please provide information on the variability of these softer materials across the footprint of the facilities, and describe any potential impact these softer materials have on soil-structure interaction and structural response of the basemat. Please provide specific criteria on assessing their impact on uniformity assessments.**





# RAI 2.5.4-3

**Section 2.5.4.2.3.3 of the FSAR discusses the dynamic properties of rock and soil, but only discusses shear wave velocity and damping properties, and indicates that these were determined from the geophysical program. . Please provide additional information about how material damping was measured for both S- and P-wave velocities, and how material hysteretic damping was determined for site materials for both the shallow and deep velocity profiles.**

## RAI 2.5.4-4

**Calculation No. TXUT-001-FSAR-2.5-CALC-004 "Engineering Stratigraphy" indicates measured variability of the stratigraphic profile in the vicinity of the power block structures. Please provide additional information to demonstrate that this variability is within the range associated with the uniformity assumptions made in the site response and soil-structural interaction analyses conducted to estimate seismic response.**



Comanche Peak Units 3 & 4

## RAI 2.5.4-5

**Appendix D, "Spacing and Depth of Subsurface Explorations for Safety-Related Foundations," to Regulatory Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants," Revision 2 (October 2003), provides guidance for site exploration plans for safety-related foundations. One of the recommendations suggests spacing one principal boring, which is used to explore site soil or rock strata and define the site geology and the properties of the subsurface materials, per 30 m (100 ft) for tunnel or essentially linear structures. Figure 2.5.4-202 of the FSAR illustrates the exploration locations. For the west side Essential Service Water Pipe Tunnel (ESWPT) of both Units 3 and 4, the figure indicates a couple of boring locations on the side east of the structures. However, the proposed borehole is neither within the footprint nor on the side west of the structures. Taking into consideration the complexity of anticipated subsurface conditions, please explain why there is not a boring location within the footprint of west ESWPT for both Units 3 and 4.**

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Comanche Peak Units 3 & 4

## RAI 2.5.4-6

**FSAR Section 2.5.4.2.2.2.16 "Laboratory-Based Shear Wave Velocity" mentions that laboratory measurements of shear wave velocity on relatively undisturbed samples of shale, limestone and sandstone were performed. This section indicates that this testing was performed to determine the rock's degree of disturbance. FSAR Figure 2.5.4-238 provides Laboratory Shear Wave Velocity measurements vs. elevation. Given the large degree of variability in shear wave velocities encountered in the limestone layer, please discuss how this meets the uniformity criteria mentioned in FSAR Section 2.5.4.2.**

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Comanche Peak Units 3 & 4

## RAI 2.5.4-7

**FSAR Subsections 2.5.4.2.2.2.5 and 2.5.4.2.3.4.4 state that the organic content of specimens was determined in general and the test results are provided in the Laboratory Test Data Report. Please clarify whether any test results for undocumented fill are included in these test results. In addition, were any tests for chemical properties performed to determine chemical contents of the undocumented fill, such as pH value, chlorides, sulfates, etc? Please provide information on these chemical contents, and assess the potential impact on the groundwater chemicals due to these chemical contents.**

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## RAI 2.5.4-8

**TXUT-001-FSAR 2.5-CALC-003 “Shallow Velocity Profile Development-Slope Method,” Page 8, indicates that no velocity measurements were taken from depths 415 ft to 465 ft. In this region, the velocities are inferred from other data.**

1. **Please explain why the variability in properties for this region is not increased, since the velocities are not based on measurements.**
2. **On the basis of the lack of actual measured data, explain why the apparent larger uncertainty associated with this portion of the profile is or is not reflected in increased variability of the design velocity profile in this section, as opposed to the level of variability one would expect when using the maximum range from the measured data.**
3. **In the alternative, demonstrate quantitatively, that there is good correlation between the parameters used to extend the measured velocities and the actual measured velocities.**





## RAI 2.5.4-9

**Sections 2.5.4.1.5 and 2.5.4.5.1 of the FSAR indicate that the power block structures are set back from the top of the reservoir slopes about 150’ to 200’, and that no evidence of previous landsliding has been found. Please clarify whether there are any assessments for the adequacy of this standoff distance to provide sufficient support for soil-structural interaction and lateral sliding. Also, please provide the specific evaluations performed to indicate that this standoff distance has been taken into account, and identify whether there have been any impacts to the stability evaluation for facilities of the plan area of the power blocks.**

# RAI 2.5.4-10

**FSAR Figure 2.5.4-217 shows a general conceptual excavation cross-section. Please describe the procedure that will be followed during site excavation and construction activity to ensure that appropriate strata for proposed foundation locations, as described in the FSAR, are confirmed through objective measures and the exposed foundation laying surface is uniform. Any part of the contact surface of foundation that is shale and not Glen Rose limestone, should be removed and the remedial measures should be described in the FSAR. Please provide vertical and horizontal extent of all seismic categories I excavations, fills, and slopes, including the locations and limits of excavations, fills, and backfills on plot plans and geologic sections and profiles.**





# RAI 2.5.4-11

- **Subsection 2.5.4.5.1.2 in the FSAR proposes that concrete fill will be used for foundation preparing, and further states that the fill concrete has a design compressive strength of 3,000 psi to meet the strength requirement. Please address the concrete durability, as described in American Concrete Institute (ACI) 201.2R, for fill concrete.**
- **Erosion of porous concrete sub-foundation, as described in NRC Information Notice (IN) 97-11, and leaching of calcium hydroxide could be potential problems, since the assumed water ground table (EL. 780 ft) is very close to proposed approximate excavation bottom (about EL. 782 ft), and even could be higher than some localized excavation areas, which need to be deepened below EL. 782 ft to remove disturbed or unstable material. In addition, ground water and perched water seeping down along the sides of the structures could cause potential impact on porous concrete fill. Please explain how the differential settlement due to erosion, and loss of concrete strength due to leaching, will be addressed, and provide justification for the manner in which these potential issues will be addressed.**

## RAI 2.5.4-12

By letter dated April 2, 2009, Luminant provided a revision to FSAR Subsection 2.5.4.5.4.1.2 “Fill Concrete.” In its revision, Luminant proposed using the American Society for Testing and Material (ASTM) C94/C94M-07 “Standard Specification for Ready-Mixed Concrete,” for use of ready mixed concrete for backfill purposes. The bulk of the ASTM C94/C94M standard is a performance, or end-result, specification. ASTM C94/C94M does not prescribe a method of achieving these requirements and results, such as how to achieve the slump, the air content, the temperature, or minimum strengths. Please indicate why the ASTM C94/C94M standard, and not the standard in American Concrete Institute 349, will be used.





## RAI 2.5.4-13

Although the backfill material sources have been identified as excavated limestone in FSAR Section 2.5.4.5.4, please discuss the steps that will be taken to avoid inclusion of shale, or other undesirable material, which is unsuitable for structural backfill.

## RAI 2.5.4-14

**Table 4 of TXUT-001-PR-007, "Dynamic Profile," provides dynamic properties of subsurface rock materials. Please clarify the following:**

a. **Table 4 refers to Curve 1, 2, 3, 4, 5, Figures 1 or 2 for the relationship describing the variation in shear modulus and damping to strain. Please include these figures in the document.**
b. **Table 4 indicates that the Cv used for upper bound (UB) and lower bound (LB) are not the same for some layers, which would indicate that some distribution other than lognormal is used for the shear moduli. What is the basis for use of the non-log normal distribution for soil/rock properties?**
c. **Note 11 of Table 4 refers to Figure 2b for damping. Please include this figure in the document.**
d. **Subnotes C and D of Table 4 indicate that the damping has been adjusted downward for use in development of the ground motion response spectra (GMRS). Please reflect this change in the text.**





## RAI 2.5.4-15

**FSAR Section 2.5.4.8 states "Thus, the engineered compacted fill does not meet the conditions stated in RG 1.206 or RG 1.198 that would cause suspicion of a potential for liquefaction, and no liquefaction analysis is necessary. Even in the unlikely event that the engineered compacted fill became completely saturated, the soil density is too high and the site PGA range is too low to suspect a potential for liquefaction". Please provide a quantitative comparison to validate the statement, given that some fill material will be granular. Also, please provide an analysis to verify the effect of potential liquefaction of duct banks and buried safety related piping and tunnels.**

## RAI 2.5.4-16

**Please provide a reference to the appropriate section in Chapter 19 where seismic margins analysis for site specific soil liquefaction and bearing capacity with respect to an earthquake of 1.67 times the Safe Shutdown Earthquake are demonstrated.**





## RAI 2.5.4-17

**Section 2.5.4.10.1 of the FSAR indicates values of ultimate bearing capacity. Calculation No. TXUT-001-FSAR-2.5-CALC-009, "Settlement and Bearing Capacity," indicates that these were determined from standard formulae associated with static load conditions. The statement is made (FSAR page 189) that the ultimate bearing capacity of the Glen Rose Formation is 146 ksf. Please provide information on how dynamic effects were included in the assessment of ultimate bearing capacity, compare the ultimate bearing capacity with dynamic bearing demand, and assess safety factors under dynamic loads.**

# RAI 2.5.4-18

**Section 2.5.4.10.5 of the FSAR indicates that resistance to lateral loads can be achieved by both passive soil pressure as well as friction below the base. Please provide information on how safety against sliding was computed incorporating consistent displacement estimates for both friction under the basemat and passive pressure estimates. Please provide information on how ultimate friction coefficients were computed between basemat and fill materials potentially located under the basemat.**





# RAI 2.5.4-19

**FSAR Section 2.5.4.10.4 "Lateral Earth Pressure" reference FSAR Figure 2.5.4-242-2.5.4.-243 which provides calculation of the lateral active and at-rest pressures for selected granular backfill. Please provide sample calculations considering effects of the seismic lateral earth pressure on the retaining structures.**

## RAI 2.5.4-20

**Calculation No. TXUT-001-FSAR-2.5-CALC-009, "Settlement and Bearing Capacity," indicates that the 50th percentile ultimate strength of the shale material is approximately 10 to 15 tsf, while the dynamic demand under the reactor building (static plus seismic loads) is over 30 tsf. The dynamic demands under the other facilities are also high, relative to this ultimate material strength. Please provide information to indicate that the shale material, as well as other such low-strength materials, will not be found under the power block facilities, and the program that will be used for confirmation.**



Comanche Peak Units 3 & 4

## RAI 2.5.4-21

**FSAR section 2.5.4.10.2, "Settlement," states that "settlement estimates are based on interpreted compressibility characteristics and elastic modulus properties of Glen Rose Formation limestone and shale materials, as discussed in Subsection 2.5.4.2." Please provide the settlement monitoring program that will be used during and after construction.**

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## RAI 2.5.5-1

**FSAR Section 2.5.5.2.5. states that a pseudo-static method was used for the slope stability analysis at the site. The guidance described in SRP 2.5.5.2 specifies that both vertical and horizontal motions be considered in the evaluation of slope stability. Demonstrate how the vertical motion was considered in the slope stability analyses.**





## RAI 2.5.5-2

**FSAR Figure 2.5.5-210 presents the static stability analysis for Cross Section E-E'. The slope stability failure surface indicated in this Figure appears to be pushed up above the retaining wall. Indicate whether the Factor of Safety is dependent upon the capacity of the wall. Please provide a description of the design of this wall.**

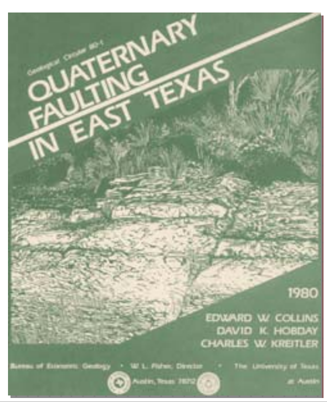
# RAI 2.5.5-3

**FSAR Subsections 2.5.4.1.5 and 2.5.5.1.2 indicate that localized surficial erosion and raveling have occurred in undocumented fill and/or native colluvial soils on the reservoir slopes, and conclude that this is a surficial condition that does not present a significant slope stability hazard to the CPNPP Units 3 and 4 plant sites. Please provide information including (1) to what extent the "localized surficial erosion and raveling" has happened, (2) the technical basis of the applicant's conclusion that there is no significant slope stability hazard, and (3) what, if anything, the applicant intends to do to ensure the maintenance and protection the slope for CPNPP Units 3 and 4. In addition, please explain whether this local erosion and raveling is considered as a factor in the slope stability analyses presented in Subsection 2.5.5.3.**

## Mt. Enterprise - Elkhart Graben (MEEG)

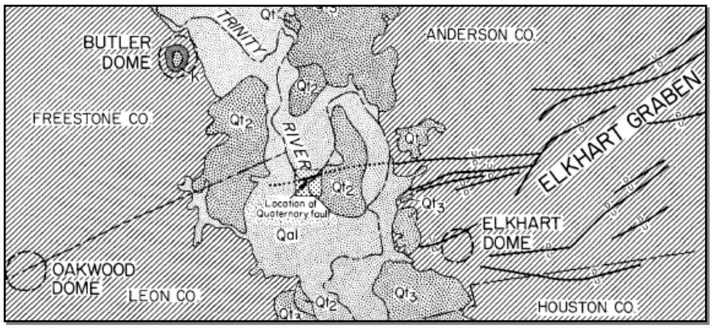
- Discussed in detail in Subsection 2.5.1.1.4.3.4.2
  - Late Jurassic to Early Cretaceous normal fault system
  - Youngest offset units are Eocene in age
  - Faults are rooted in Jurassic Luan salt at maximum depth of 4.5 to 6 km


- Research on MEEG comes from studies predating EPRI-SOG (i.e., pre-1986)
- Dominant opinion is that MEEG is a salt-rooted structure, and any Quaternary deformation is likely related to salt migration (e.g., Ewing, 1991; Ferguson, 1984; Jackson, 1982; Murray, 1964)
- One study with results presented in grey literature suggests there has been Quaternary slip on MEEG faults (Collins et al., 1980)



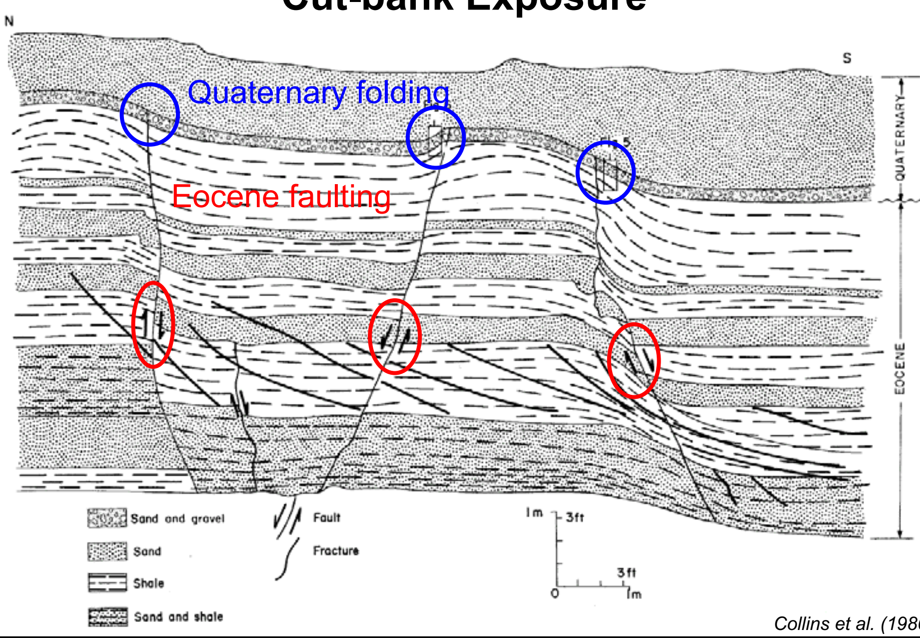
• Collins et al. present three lines of evidence they use to support their hypothesis that MEEG has Quaternary activity

- – Folded Quaternary gravels observed in river cut-bank deposits
- – presumed folded Quaternary gravels in an auger profile
- – leveling data showing down to the south change in surface elevation



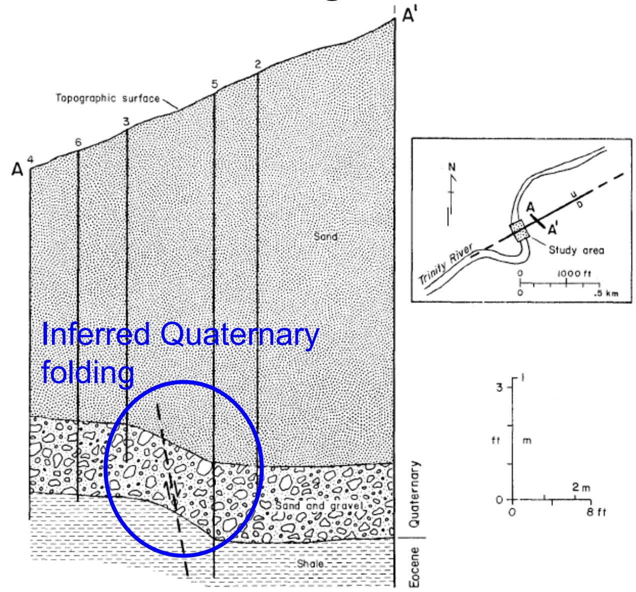

*Collins et al. (1980)*

## Folded Quaternary Deposits: Cut-bank Exposure




*Collins et al. (1980)*

## Folded Quaternary Deposits: Power-auger Holes



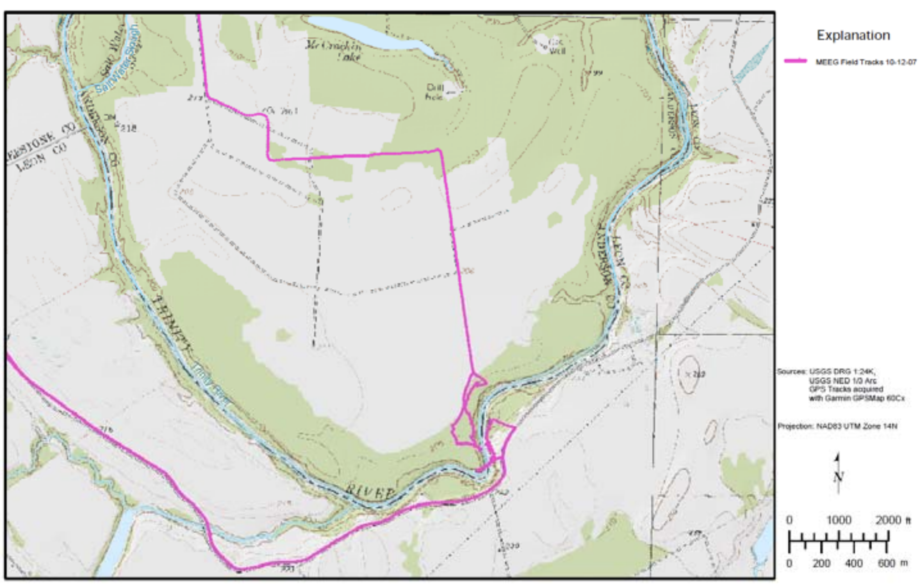

*Collins et al. (1980)*

## MEEG Capability

- Concluded not a capable fault based on:
  - – Shallow, salt driven deformation mechanism
  - – No evidence of seismogenic rupture
    - Potentially folded Quaternary deposits show no evidence of scarp formation
    - No observed or reported scarps
  - – Potential evidence for slow, aseismic creep
  - – Research on MEEG was pre-EPRI-SOG, so no new information to suggest inclusion as a source or a modified evaluation of capability

# MEEG Field Reconnaissance

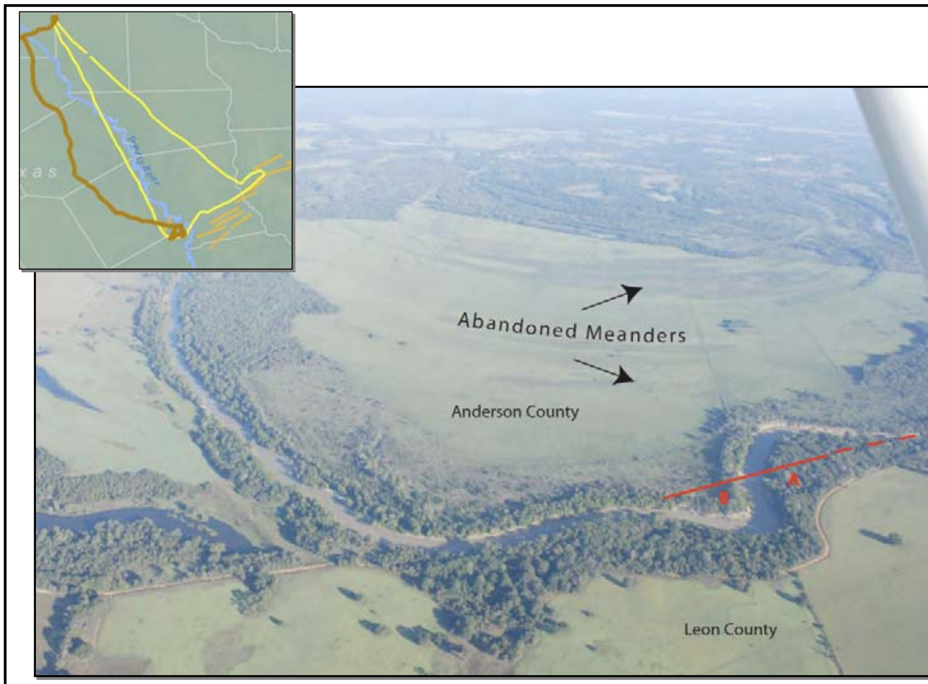
- Despite strong conclusion that MEEG was not capable, performed ground and aerial reconnaissance of Collins et al. field areas


A


C


D

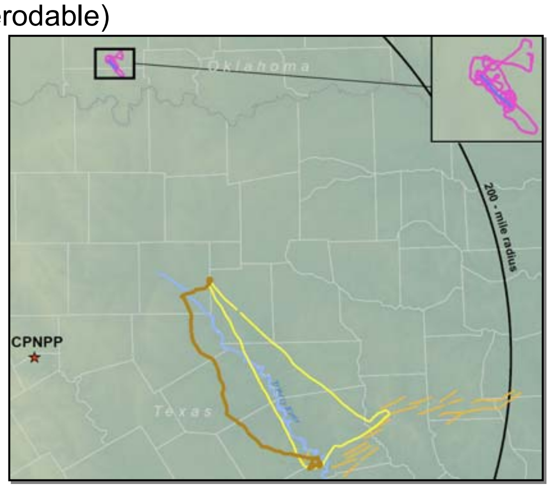

Abandoned Meanders
Anderson County
Leon County

## MEEG Field Reconnaissance - Conclusions

- Faulted Eocene rocks and potentially folded Quaternary sediments of Collins et al. (1980) are contained within a large river-bank slump and cannot be used as constraints on faulting
- No evidence of folded Quaternary sediments was observed
- Therefore, no evidence to suggest MEEG is a capable fault

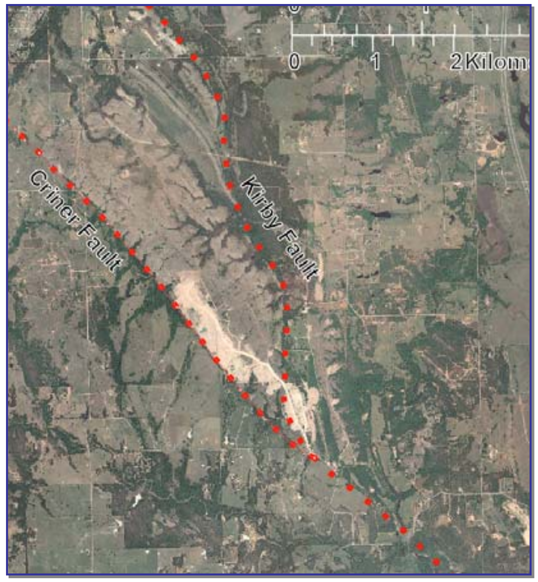
## Criner Fault

- Discussed in detail in Subsection 2.5.1.1.4.3.4.2
  - Distinct fault-line scarp caused by differential erosion of the Ordovician limestone of the Criner Hills (resistant) and the Pennsylvanian limestone, sandstone & shale of the Marietta Basin (erodable)



## Criner Fault

- Potential for Quaternary deformation noted in unpublished consultant reports by Geomatrix (1990, 1993)
- Primarily based on observation of sheared Pleistocene to alluvium
- Later studies demonstrated shearing was due to landslide (Williamson, 1996; Hanson et al., 1997)

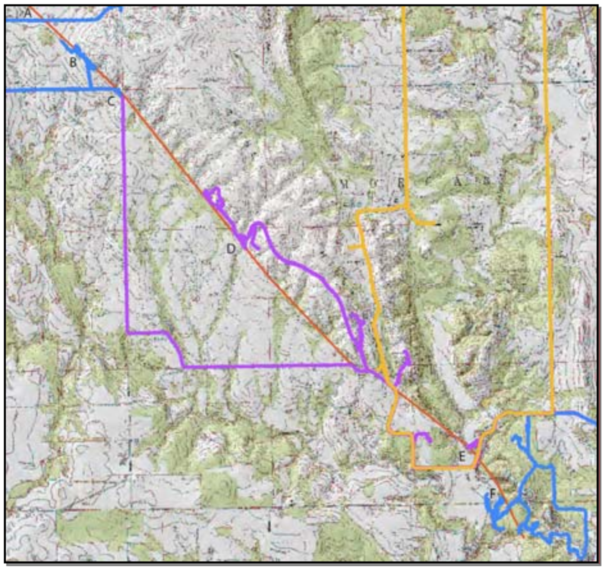



## Criner Capability

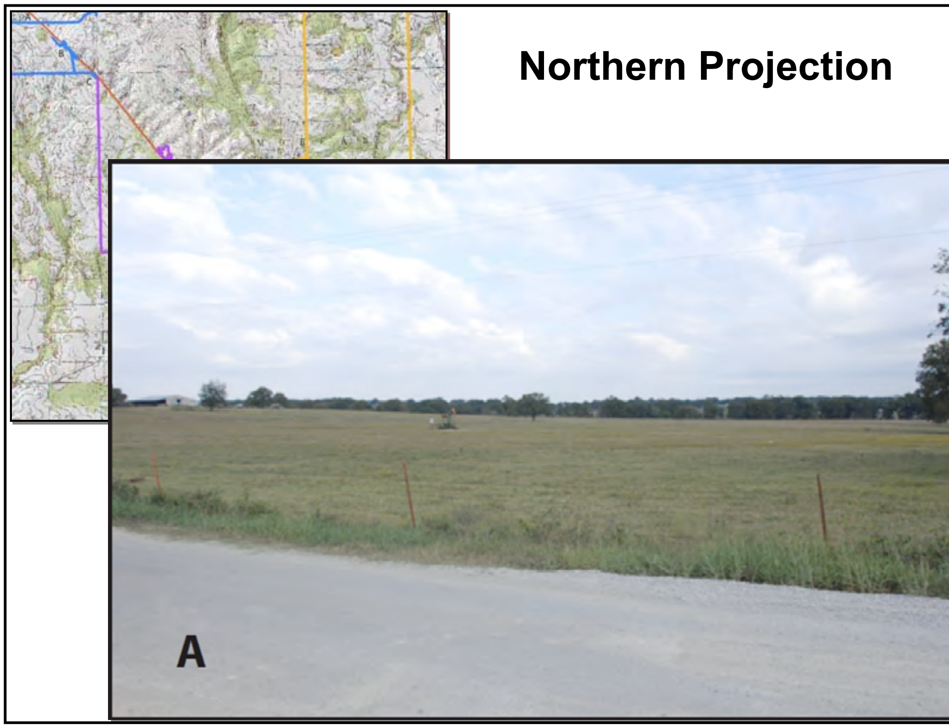
- Concluded not a capable fault based on:
  - No evidence of Quaternary activity
  - Previously hypothesized evidence was weak and has been reinterpreted

## Criner Field Reconnaissance

• Despite lack of Quaternary evidence of activity, ground and aerial reconnaissance conducted

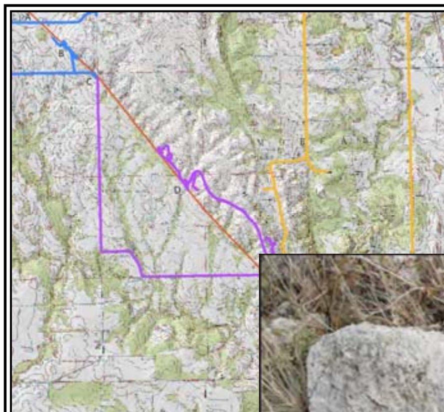


## Northern Projection

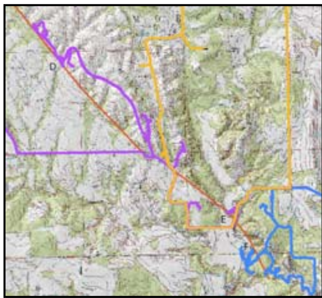


## North Criner Hills

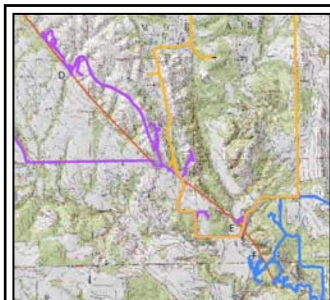

## Potential “Fault Plane”



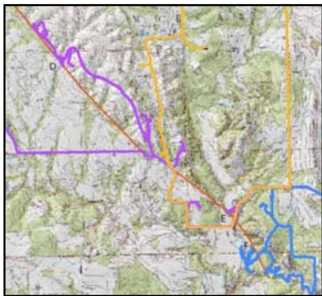
## Outcrop with Hypothesized Shear Zone

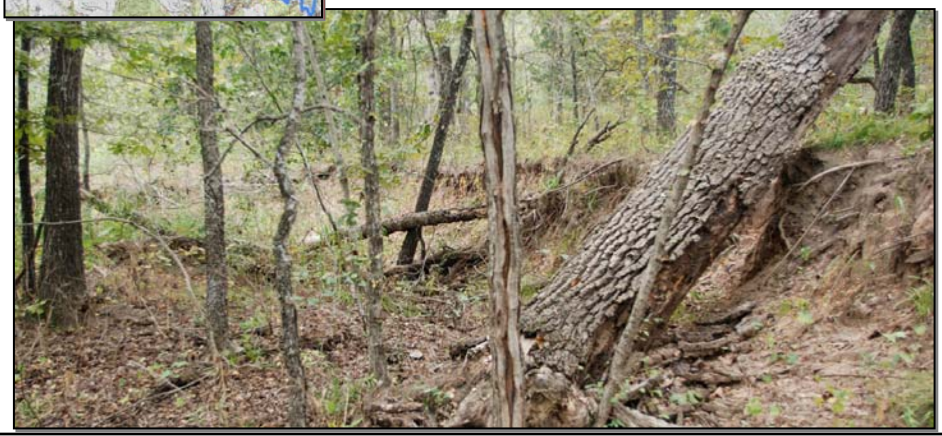


## Outcrop with Hypothesized Shear Zone



## Landslide Headscarp





## Criner Field Reconnaissance - Conclusions

- Hypothesized Pleistocene shearing is correlated to landslide
- No evidence of Quaternary faulting
- Therefore, no evidence to suggest Criner is a capable fault