May 28, 2020 GSI Project #: 200320

Mark Lancaster
Program Manager
Five Counties Salmonid Conservation Program
mlancaster@5counties.org

Subject: Geotechnical Investigation
Forest Avenue Bridge and Lee Fong Park
Weaverville, CA 96093

Dear Mr. Lancaster:

In accordance with your request and authorization, GeoServ, Inc. (GSI) has prepared the enclosed Geotechnical Investigation based on the requirements and proposed project specifics identified during our review. Specifically, the report provides bridge foundation improvement design recommendations and other stream channel restoration construction recommendations for design and construction of this project in Weaverville, California.

Implementation of this stream channel restoration project at this location is considered feasible from a geotechnical standpoint provided the recommendations contained in the attached report are incorporated into the design. If you have any questions regarding our findings or recommendations, please do not hesitate to contact this office. The opportunity to be of service is appreciated.

Respectfully submitted,

James Fitzgerald, Principal
GeoServ, Inc.
624 S. Mt. Shasta Blvd.
Mt. Shasta, CA 96067
530.227.8963
jf@geoscienceserv.com

Forest Avenue Bridge and Lee Fong Park Geotechnical Investigation, Weaverville, California

Prepared for: Five Counties Salmonid Conservation Program

Prepared by: GeoServ, Inc.

Initial Report Date: May 28, 2020

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Introduction

This memorandum documents the methods, settings, and results of a geologic and soils investigation completed for the Forest Avenue Bridge over Weaver Creek and Lee Fong Park Stream Restoration project(s) repeater for the Five Counties Salmonid Conservation Program (5C's). The projects are located near in the town of Weaverville, Trinity County, California (Figure 1). Field investigation and existing data were used to classify and characterize the engineering properties of the project area rock & soil to evaluate potential designs and construction techniques.

This investigation completed the following:

1. General description of geologic setting and geologic hazards.
2. Type of soils anticipated to be encountered during excavation.
3. Temporary (i.e. OSHA soil type) and permanent cut-slopes for stability.
4. Forest Avenue bridge improvement geotechnical design recommendations and specifications.
5. Suitability of soils for reuse on existing fields in the Lee Fong Park and within the existing Weaver Creek stream channel and recommendations for placement and compaction of fill material.

Investigation Methods

This investigation was completed to obtain information on the engineering properties of the rock, soil, groundwater, and to evaluate potential designs and construction techniques. The engineering properties of the site rocks and soils were assessed using industry standard methods (Cruden and Varnes 1996, BOR 2001, U.S. Army Corps of Engineers 1960, and USDA 1994). The rocks and soils were classified and assessed following the most recent ASTM methods.

At the Forest Avenue Bridge, the investigation included two (2) boreholes and site survey of bedrock geometry. The boreholes were located in the near the Forest Avenue bridge over Weaver Creek. A Ground Penetrating Radar (GPR) Survey was conducted at the bridge location.

Within the Lee Fong Park project area, a GPR survey was performed in conjunction with five (5) hand dug test pits.

The subsurface explorations (drilling, test pits & GPR) scheme was intended to assess the horizontal and vertical distribution of fill, soil, rock, and groundwater near in the proposed project area(s). For each borehole and test pit, the rock/soil depth, color, particle size and volume, relative density, particle angularity and shape, moisture content, strength, cohesion, and compaction were visually noted and field classified.

A Standard Penetration Tests (SPT) were completed at depth intervals where possible. Generally, SPT samples are taken incrementally of drilled depth or at specific depths (e.g., soil type change) to help measure and quantify the relative density and consistency of the soil. Sampling of drill cuttings of rock samples were taken where possible. Incremental SPT sampling was limited due to a shallow depth to rock. SPT tests were completed following ASTM 1586. Split spoon core samples were collected, photographed, and field classified. The recovery of un-disturbed samples was limited given the material characteristics. The borehole logs are shown in Attachment A.

Geologic and Tectonic Setting

The rocks and soils that underlie the project area are of the Klamath Mountains Geomorphic Province (Figure 1) (Jennings 1977). The regional and local topography are an expression of these relatively old marine sedimentary and metasedimentary deposits. As mapped in the USGS Redding Quadrangle (OFR 2012-1228) the site is underlain by the Quaternary Alluvium (Qa) this formation is characterized by locally derived sediments associated river erosion/deposition along Weaver Creek. The rocks mapped and intercepted in drill holes advanced for this investigation match the Quaternary Alluvium characterization. It should be noted that Quaternary Alluvium encountered at the site may have been placed through previous mining or grading activities along the creek.

Determination of actually placement mechanisms of the material (i.e. natural, mining, construction related) at the site is an estimate and is difficult to determine.

Results

Subsurface Conditions

There are no exposed rock outcrops within both project areas. USGS mapped both areas as Quaternary Alluvium (Qa). This formation is characterized by locally derived sediments associated river erosion/deposition along Weaver Creek. The NRCS Soil Survey Staff 2020 soil report mapped the soil type as 102-Atter-Dumps, Dredge Tailings-Xerofluvents Complex, 2 to 9 percent slopes that is somewhat excessively drained and has a fill component associated with dredger tailings. Given the known historic mining and construction at the site it is assumed that most loose or moderately dense material at the site and to depth are derived from dredge tailings and are in fact artificial fill. The area is likely underlain by nonmarine (continental) sedimentary rocks, Oligocene, sandstone, shale, and conglomerate; mostly well consolidated (Figure 1).

Forest Avenue Bridge

The bridge project area is about 0.1 acres and drains in general toward Weaver Creek and intern to the south southeast, with moderate slopes that increase in gradient along the creek bank areas. Bridge concrete abutments are protected by use of concrete/shotcrete embankments that extend upstream and downstream of the bridge with the majority of the creek bottom covered multiple pores of concrete that are in varying levels of disrepair. The bridge is of a single span with multiple steel beams for the span and a concrete/asphalt deck. Construction of the bridge appears to have occurred as at least two phases with the downstream 2/3rds of the bridge being constructed prior to the upstream 1/3. These two separate construction times are evident in the cutting into the bridge abutments and later concrete pours for the placement of the upstream 1/3 steel beams. Timing of when these two events took place is unknown, but available aerial photos show a relatively thin width in the current bridge location. Also timing of the placement of instream features such as the shotcrete embankments and the stream bottom concrete is unknown. The site is vegetated by seasonal grasses and brush, with several mature trees in the areas up and downstream of the bridge.

Drill holes at the site were advanced on April 30, 2020 and May 1, 2020 using a rotary auger drill (Figure 2). Drilling consisted of an 8 in diameter diamond coring of asphalt followed by 6 in hollow stem auger drilled to practical refusal, and then a mud rotary tri-cone drilling a 3 in diameter hole to the full depth of the subsurface exploration. DH-1 and DH-2 located on the east and west of the Forest Avenue Bridge over Weaver Creek respectively, those drill holes correlated well below the ground surface (bgs) (see Figure 2 and Attachment A for drill hole logs). Both drill hole had about 8 in of asphalt at the surface, with aggregate base material extending below to a depth of 1.5 ft in DH-1 and 0.8 ft in DH-2. Below the aggregate base in each drill hole a layer of artificial fill consisting of Sandy Gravel with Cobbles and Boulders (GW) extended to a depth of 3.5 ft in DH-1 and 3.25 ft in DH-2. Below the artificial fill drilling became more difficult and a Clayey Gravel with Cobbles and possible Boulders were encounter to the total depth of the drill holes. This Clayey Gravel was brown, moist to wet, moderately to very dense with low to medium plasticity clay, subangular to angular gravel less than 1.5 in, cobbles less than 8i n and boulders up to 24 in. This lowest unit is interpreted as being native soil in this report but may have been placed during mining or construction operations of an unknown time period. The holes likely penetrated into the upper portion of the Weaverville Formation with reddish brown cutting return and tri-cone drill refusal.

The GPR survey at the Forest Avenue Bridge was conducted May 3, 2020 using a Mala Geoscience shielded 250 megahertz (MHz) GPR antenna/receiver skid. Four profiles were completed, their location can be seen on Figure 2 (see Attachment A for GPR Survey Logs). Data collected by the antenna/receiver was stored and quality control checked using a Mala GX Controller in the field. Data was downloaded and further reviewed using Mala Object Mapper software. Overall GPR data correlates with drill hole and other data created for this study.

Lee Fong Park

The Lee Fong Park stream restoration area is about 4.3 acres and drains in general toward Weaver Creek and intern to the south in the stream channel, with moderate slopes that increase in gradient along the creek bank areas. The site is vegetated by seasonal grasses and brush, mature trees along the project area length. Previous grading has occurred along the length and on both banks of the creek. That grading incudes park improvements (trails & roads), car parking areas, sewer/stormwater pipelines (along and across/under the stream), placement of riprap or rubble for erosion control in the stream and other ancillary improvement.

The GPR survey for the Lee Fong Park area was conducted May 3, 2020 and May 26, 2020 using a Mala Geoscience shielded 250 megahertz (MHz) GPR antenna/receiver skid. Four profiles were completed, their location can be seen on Figure 3 (see Attachment A for GPR Survey Logs). Data collected by the antenna/receiver was stored and quality control checked using a Mala GX Controller in the field. Data was downloaded and further reviewed using Mala Object Mapper software. Overall GPR data correlates with test pit and other data created for this study.

The test pits were hand dug on May 3, 2020 and were located near the GPR Survey line locations to assist in GPR survey interpretation and to assess site in-place rock and soil characteristics. Test pit locations are shown on Figure 3. Test pit logs are listed in Table 1.

Table 1. Hand dug test pit logs.

Test Pit ID	Depth	Soil Description
TP-1	0-2.0	Silty Sand (SW) – light brown, dry to moist, moderately dense, silty low plasticity, sand very fine to coarse.
TP-2	0-1.5	Gravelly Sand with Cobble (SW-GW) – grey sand, blueish grey oversize, dry, moderately dense, sand fine to very coarse, gravel less than 1.5 in diam. sub-rounded to rounded, cobbles less than 6 in diameter sub-rounded to rounded
TP-3	0-1.5	Silty Sand (SW) – light brown, dry to moist, moderately dense, silty low plasticity, sand very fine to coarse.
TP-4	0-1.5	Gravelly Sand with Cobble (SW-GW) – grey sand, blueish grey oversize, dry, moderately dense, sand fine to very coarse, gravel less than 1.5 in diam. sub-rounded to rounded, cobbles less than 6 in diameter sub-rounded to rounded
TP-5	0-2.0	Gravelly Sand with Cobble (SW-GW) – grey sand, blueish grey oversize, dry, moderately dense, sand fine to very coarse, gravel less than 1.5 in diam. sub-rounded to rounded, cobbles less than 6 in diameter sub-rounded to rounded

Subsurface mapping and field soil classification data indicate that the Lee Fong Park area has relatively homogenous native soil within the project area. The soil consists of Gravelly Sand with Cobbles and Silty Sand. These soils extended to the full depth of test pits advanced for this study. These materials are expected in a location near the stream, and they are expected to underlay areas in the vicinity of test pits and GPR survey lines. Although in-place soils appear to be homogenous, artificial fill at the site varies greatly. Limited mapping of areas with surficial artificial fill can be seen on Figure 3. Artificial fill at the site includes materials place for parking areas, trails, and erosion control measures in and along the banks of the stream. Artificial fill material used in off-stream grading was typically well graded gravels while instream and bank fills consisted of riprap up to 24 in in diameter as well as concrete fragments up to 48 in in diameter. These instream features appear to be associated with storm water/sewer pipeline that exist under the stream and area likely used to prevent erosion down to those pipeline elevation(s)

Groundwater Conditions

Groundwater was not encountered within the drill holes drilled for this investigation. Groundwater elevations can fluctuate from season to season and year to year, and as such groundwater may pose a problem requiring mitigation at the time of construction of the project as well as post construction. If groundwater and or seepage is encountered during construction, mitigation techniques need to be developed by the construction contractor to allow for dewatering of affected areas. If groundwater or seepage is encountered during construction GeoServ will need to be contacted in order to assess its effect of the final design of the structure and any possible negative affects it may have over the lifetime of the structure. Due to the close vicinity of both sites to Weaver Creek it is expected that groundwater will affect the construction phase of any project occurring at this site.

Geologic Hazards and Seismic Consideration

Fault Rupture and Seismic Ground Shaking

Based on the distance from known active faults, the risk of surface fault rupture is low. The magnitude of seismic ground shaking that could affect the proposed structure(s) was estimated using the historic seismic events and seismic hazard assessment following CGS guidelines.

Historic Seismic Events

Historically and presently, the region to the west of the project area has been subject to fault activity. Figure 1 shows the location and distribution of the Alquist-Priolo Fault Zones Map and Holocene faults (i.e., active faults). The Richter magnitude scale is used to quantify the amount of seismic energy released by an earthquake. Earthquakes with a magnitude greater than three can be felt by most people, but significant damage usually only occurs in earthquakes that have a magnitude greater than five (USGS 1989). Several earthquakes with a magnitude of five or greater, have had an epicenter within 100 miles of the project area in the last 100 years. However, the region has regularly experienced localized smaller magnitude (between three and five) earthquakes over the last 100 years within 50 miles of the project area. The Alquist-Priolo (CDC 2002) Earthquake Fault Zone

Active Faults and Seismic Hazard Assessments

Project construction and implementation would be subject to a low to moderate risk of damage from fault movement. Fault movement has the potential to affect the stability of the proposed structure(s). According to the CDC (2000), the closest known inactive fault is approximately 48 miles east of the project area (Figure 1). Most of the faults east of the project area are considered active, and the most recent events with epicenters near McArthur and Burney were 4.3 and 4.4 magnitude earthquakes in 1974 and 2005, respectively. To initiate the dominant seismic hazards of the area, an earthquake would have a magnitude of 8.5 or greater (CDC, 1996).

Seismic movement from earthquakes has the potential to affect the stability of the proposed structure(s). According to the CDC (1997) and CDC (2006), the project area is not within a mapped Alquist-Priolo Earthquake Hazard Zone. It is likely that the proposed structure(s) will be impacted by the effects of a large magnitude earthquake due to proximity to known active fault zones. The proposed structure(s) will likely be subjected to frequent smaller magnitude earthquakes. Small earthquakes may cause minor settling or shifting of unconsolidated sediments. Overall, there is a low to moderate risk of damaging earthquakes (Peterson 1996, Peterson 1999, and Toppozada, 2000).

Liquefaction

Liquefaction typically occurs as a result of seismic events that cause the sudden loss of soil shear strength. The cyclic loading from an earthquake triggers liquefaction. The risk of liquefaction is based on the expected seismic event, soil properties, and groundwater depth. For liquefaction to occur the following must be present:

Granular soils;

Low soil density; and
High groundwater table.

The project area rock or soils are granular in nature and lie atop dense oversized granular soil (cobbles & boulders). The risk of adverse impacts from liquefaction at the project area is low if the foundations are located in dense oversized granular soil.

Expansive Soils

Potentially expansive clay soil was not encountered at the site. The risk of expansive soils is low.

Volcanic Hazards

The project area is not within an area with recent volcanic activity, and the project area is in a zone that could be impacted by a volcanic eruption. Quantifying the volcanic risk to the project area is beyond the scope of this inspection. The project area does not fall within a volcanic risk area according to the USGS. The risk of adverse impacts from volcanic activity at the project area is low.

Slope Stability

The project area is within a region with high landslide susceptibility. Based on the site location and topography and subsurface geology there is a low to moderate modern landslide risk.

Tsunamis and Seiche

Based on site location, elevation, and tsunami hazard mapping from the CGS website (http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=tsunami) the site is not in a tsunami inundation hazard zone. In addition, oscillatory waves (seiches) are considered unlikely due to the absence of large confined bodies of water in the site area.

Erosion

There is a high erosion risk given the work that will need to be done at the top of the Oregon Mountain with moderate to steep slopes moving away from the site that runoff into several creeks/headwaters. Any construction related disturbance to the soils will increase the erosion risk, and temporary and permanent erosion control measures should be implemented to keep stormwater from discharging site soils into the creek. A dewatering plan will be needed for the installation of each abutment.

Conclusions

General Earthwork and Grading

Site Preparation

Areas to be graded should be stripped of vegetation, organic debris, concrete, and/asphalt. These materials should be properly disposed of offsite and kept out of fill material. Rocks greater than 8 in diameter should be separated from the fill material before the fill is placed and compacted.

Excavations: Trenching Hazards

Given the measured soil and rock properties, for excavations that are less than 20 ft bgs and have no groundwater present, the slopes should be no steeper than 1H:1V. Trenches with groundwater present should be no steeper than 3H:1V. OSHA categorizes soil and rock deposits into four types, A through D. Measurements taken during the investigation indicate that there will likely be two main types to include Type A with an unconfined compressive strength greater than 1.5 tons per square foot (tsf) and Type B Soils an unconfined compressive strength greater than 0.5 tsf but less than 1.5 tsf. The soils tested tend to moderately dense. Trenches deeper than 10 ft should be evaluated by the Design Engineer or Geotechnical Consultant to confirm trench stability. Exposed soil should be protected from erosion and temporary excavation slopes angles and/or trench shoring are required to meet OSHA standards and are the responsibility of the contractor and should meet OHSA requirements.

Temporary spoil material cannot be placed closer than 2 ft from the surface edge of the trench or excavation. Spoil material cannot concentrate storm water run-off into the the excavation. For trenches that are 4 ft or greater bgs, ingress and egress access points should be maintained so that workers do not have to travel more than 25 ft laterally to the nearest access points. Sloping access points back to at least 3H:1V is recommended rather than using ladders.

Forest Avenue Bridge Foundations

Based on the results of the subsurface investigation, the existing bridge is likely founded on dense to very dense Quaternary alluvium and rock (Figure 4).

Vertical Loads

The safe bearing capacity of is about 4,000 pounds per square foot (psf) for abutments and concrete structures founded on firm and unyielding rock.

Lateral Loads

Horizontal shear forces are offset by frictional forces between the base of footings and the finished subgrade material. Since the subgrade is likely to be made up of very dense rock and/or soil, shallow footings may be designed to resist lateral loads using a coefficient of friction of 0.35 (total frictional resistance equals the coefficient of friction times the dead load). A design passive resistance value of 250 psf per foot of depth (with a maximum value of 1250 psf) may be used. The allowable lateral resistance can be taken as the sum of the frictional resistance and the passive resistance, provided the passive resistance does not exceed two-thirds of the total allowable resistance.

The estimated coefficient of active earth pressure (K_a) is estimated to be about 0.22 using 40^0, the estimated friction angle (ϕ') of the in-place rock at 4' bgs.

Retaining Walls

Free draining retaining walls backfilled using onsite fill may be designed using the equivalent fluid weights given in Table 2.

Table 2. Retaining wall design recommendations.

Equivalent Fluid Unit Weights (pcf)

Wall Type	Level Backfill	Slope Backfill 2:1 (horizontal: vertical)
Cantilevered Wall	35	50
Restrained Wall	50	65

Wall Backfill Material Wet Density

Material Type	Wet Density (pcf)	Notes
Native Rock	145	
Drain Rock	125	Needs to be verified once backfill source is selected
Engineered Fill	130	Needs to be verified once backfill source is selected

Live load surcharges on retaining walls should generally be equal to 1/3 of the vertical load of the traffic located within ten lateral feet of wall. Lateral pressures on cantilever retaining walls (yielding walls) due to earthquake motions may be calculated. The increment of dynamic thrust in both cases should be based on a trapezoidal distribution (essentially an inverted triangle), with a line of action located at 0.6H above the bottom of the wall. The values above assume non-expansive backfill and free-draining conditions.

Measures should be taken to prevent moisture buildup behind all retaining walls. Drainage measures should include free-draining backfill materials (e.g., drain rock), have positive drainage, and perforated drains. These drains should discharge to an appropriate location. Wall waterproofing should be as specified by the Design Engineer.

The total lateral thrust against a properly drained and backfilled cantilever retaining wall above the groundwater level can be expressed as:

$P_{AE} = P_A + \Delta P_{AE}$

For non-yielding (or "restrained") walls, the total lateral thrust may be similarly calculated using:

$P_{KE} = P_K + \Delta P_{KE}$

Where P_A = Static Active Thrust

P_K = Static Restrained Wall Thrust

ΔP_{AE} = Dynamic Active Thrust Increment = $(3/8)\ k_h\ \gamma H^2$

ΔP_{KE} = Dynamic Restrained Thrust Increment = $k_h\ \gamma H^2$

k_h = ½ Peak Ground Acceleration = ½ $(S_{DS}/2.5)$

H = Total Height of the Wall

γ = Total Unit Weight of Soil ≈ 125 pcf

Seismic Design

Table 3 and Table 4 summarize the recommended CBC seismic design parameters for the new structures. The shallow subsurface material is classified as a Site Class D. This classification is based on field observations and the measured engineering soil properties. The maximum magnitude expected to impact the site is a 7.0 earthquake that would most likely occur 48 miles from the project area (Figure 1) (see Attachment A for raw seismic data).

Table 3. Project area CBC seismic design parameters based on reference document: ASCE7-10.

Parameter	Value
Site Class	D
Distance to Seismic Source	48 mi
Spectral Response Acceleration (S_s)	0.836
Spectral Response Acceleration (S_1)	0.39
Site Coefficient (F_a)	1.165
Site Coefficient (F_v)	1.62
Max Considered Earthquake Response Acceleration for Short Period (S_{MS})	0.975
Max Considered Earthquake Response Acceleration for 1-Second Period (S_{M1})	0.632
Five-percent Damped Design Spectral Response Acceleration for Short Periods (S_{DS})	0.65
Five-percent Damped Design Spectral Response Acceleration for Short Periods (S_{D1})	0.421

Table 4. Project area CBC seismic design parameters based on reference document: ASCE7-16.

Parameter	Value
Site Class	D
Distance to Seismic Source	48 mi
Spectral Response Acceleration (S_s)	1.063
Spectral Response Acceleration (S_1)	0.519
Site Coefficient (F_a)	1.075
Site Coefficient (F_v)	null
Max Considered Earthquake Response Acceleration for Short Period (S_{MS})	1.142
Max Considered Earthquake Response Acceleration for 1-Second Period (S_{M1})	null
Five-percent Damped Design Spectral Response Acceleration for Short Periods (S_{DS})	0.762
Five-percent Damped Design Spectral Response Acceleration for Short Periods (S_{D1})	null

Structural Fill

Structural fill for the various components of bridge improvements should follow the specifications listed in Table 5 according to the type and use.

Table 5. Fill compaction table.

Type	Specifications	Compaction Recommendation	Compaction Test Type
Structural Sub-Grade	Firm and Unyielding Native Material free of debris, rocks > 4", organics	Scarified subgrade to 8" depth, moisture conditioned to within 2% optimum moisture, and re-compacted to at least 95% relative compaction or until firm and unyielding under vibratory roller	ASTM D 698
Foundation Structural Fill	Native Soil or Imported Granular Fill free of debris, rocks > 4", and organics with a Plasticity Index < 12, Liquid Limit < 35, and between 15% to 35% Passing No. 200 sieve	Placed in 12" loose lifts and compacted to at least 95% relative compaction within 2% of optimum moisture or until firm and unyielding under vibratory roller	ASTM D 698
Structural Backfill: Native Soil/Rock	Coarse Granular 1" Rock with < 2% Passing No. 200 Sieve	Placed in 4" to 6" loose lifts and compacted to at least 95% relative compaction and 2% optimum moisture or until firm and unyielding under vibratory roller	ASTM D 698

Structural Backfill: Drain Rock	Crushed drain rock shall be imported material that consists of angular, durable rock and gravel free from slaking or decomposition under the action of alternate wetting and drying, free of hazardous or deleterious material, and shall have a durability index of 40 or greater and a sand equivalent of 75 or greater.	Placed in 4" to 6" loose lifts and compacted to at least 95% relative compaction and 2% optimum moisture or until firm and unyielding under vibratory roller	ASTM D 698
Structural Backfill: Aggregate Base	Coarse Granular Base with < 2% Passing No. 200 Sieve	Placed in 4" to 6" loose lifts and compacted to at least 95% relative compaction and 2% optimum moisture or until firm and unyielding under vibratory roller	ASTM D 1557

Stream Scour

Scour potential was not assessed as a part of this investigation, scour potential should be calculated by a specialist and structural components of any bridge abutments, etc. should be found below the elevation/depth of scour calculated by the specialist. This investigation recommends including rock scour protection into the bridge abutment design and/or wing wall design.

Deferential Settlement

Deferential settlement is the tendency for native material and engineered fill material to settle at deferring rates over time when loaded with structures, foundations, or other loads. Differential settlement typically occurs when a structure is placed partially on fill and partially on native material and may cause cracking and other problematic effects to foundation/structure. When a structure is placed on both cut and fill there are two possible ways to limit deferential settlement from occurring. One of the following options should be followed:

- The entire area of the structure/foundation can be over-excavated to a depth so that when backfilled with engineered fill to final grade (planned footing bottom) the entire structure/foundation is placed on a uniform thickness of engineered fill above native soil.

or

- The foundation/footings in the area of fill extend to the depth of the native soils. This deepening of the foundation/footing can be backfilled using unreinforced concrete or "lean mix" to the planned bottom of footing elevation that corresponds with the footings resting on the "cut" area native soils.

It is up to the owner/construction contractor to determine the best solution for a given structure.

Drainage

The new structure(s) are located adjacent to Weaver Creek, drainage structure design is critical in limiting of sediment and other effluent into Weaver Creek. Drainage infrastructure should include debris catches, filtering features and other measures to limit transmission of deleterious materials and substances from contacting the water way. Drainage of the structure should discharge at least 10 feet from any portion of the foundation/footing and no ponding of water should occur within 10 feet of the structure.

Lee Fong Park Bank Soils

Bank Soil Properties

The Lee Fong Park stream restoration area is mainly moderately dense Quaternary alluvium up to 2 ft bgs. Below 2 ft bgs, the soil very dense and could not be dug by hand. The estimated soil properties are listed in Table 6.

Table 6. Stream bank soil properties.

Test Pit Log Soil Description	Average Dry Unit Weight (pcf)	Friction Angle (deg)	Saturated Unit Weight (pcf)
Gravelly Sand with Cobble (SW-GW)	130	40	145
Silty Sand (SW)	100	30	120

Cut-Slope Stability

For permanent cut-slopes in stream bank soils, the slope angle should be no steeper than 2H:1V, and erosion control measures should be implemented to help ensure long-term stability. For permanent cut-slopes in rock, the slope angle should be no steeper than 1H:1V. During construction, unusual changes in rock or soil strata should be evaluated by the Design Engineer or Geotechnical Consultant.

On-Site Fill Reuse, Placement, and Compaction

The on-site soil excavated from the stream channel is suitable to be used as non-structural fill. Top soil should be preserved as possible and stockpiled to cap the fill areas. Material larger than 8", all debris, rubble, and concrete should be removed and hauled off-site prior to fill placement and compaction. The fill should be placed and compacted in 8 in loose lifts, moisture conditioned, and compacted to 85% relative compaction (i.e., track packed). The excess soil excavated from the channel can be placed and compacted in the existing field on the east side of the park (see fill off-channel fill area in 65% design drawings in Attachment A). The fill should be placed evenly and sloped to drain at a minimum of 1% slope toward the stream channel. A minimum 10 ft buffer should be left between the edge of the riparian zone and the fill.

Excess fill is also suitable to be placed as compacted fill within the existing channel. The fill should be placed and compacted in 8 in loose lifts, moisture conditioned, and compacted to 90% relative compaction (see fill stream channel areas in 65% design drawings in Attachment A).

Construction Observation

The recommendations provided in this report are based on the sampled area and limited drill and surficial data. The interpolated subsurface conditions, on which this report relies, should be checked in the field during construction to verify conditions described herein are as anticipated. Any changes which occur to preliminary information provided GeoServ, Inc. as of the date of this report, GeoServ, Inc. should be notified and afforded an opportunity to update information provided in this report.

Recommendations provided in this report are based on the understanding and assumption that GeoServ, Inc. will provide the observation and testing services for the project. All earthworks should be observed and tested to verify that grading activity has been performed according to the recommendations contained within this report. The Project Engineer or Geotechnical Consultant should evaluate all foundation excavations.

Foundation Post Construction Settlement and Other Geologic Hazards

Given the measured soil engineering properties, the risk of measurable post-construction static settlement is low if the abutment is tied into/anchored into the outlined refusal points. As with any engineered foundation, settlement is expected to occur immediately following loading; however, settlement should be less than ¾ in over 20 ft.

Limitations

This investigation used existing data and information supplemented by field inventory data to investigate the geologic and soil engineering properties, geologic hazards, and rock/soil safe bearing capacity for the proposed foundation(s). Given the project risks, the data collection, and analysis effort are sufficient to ensure that the recommendations made herein are adequate and there will be no adverse impacts from geologic hazards evaluated herein. The confidence is high that if the recommendations are followed as specified and special inspections occur, then the risks of fill settlement is low.

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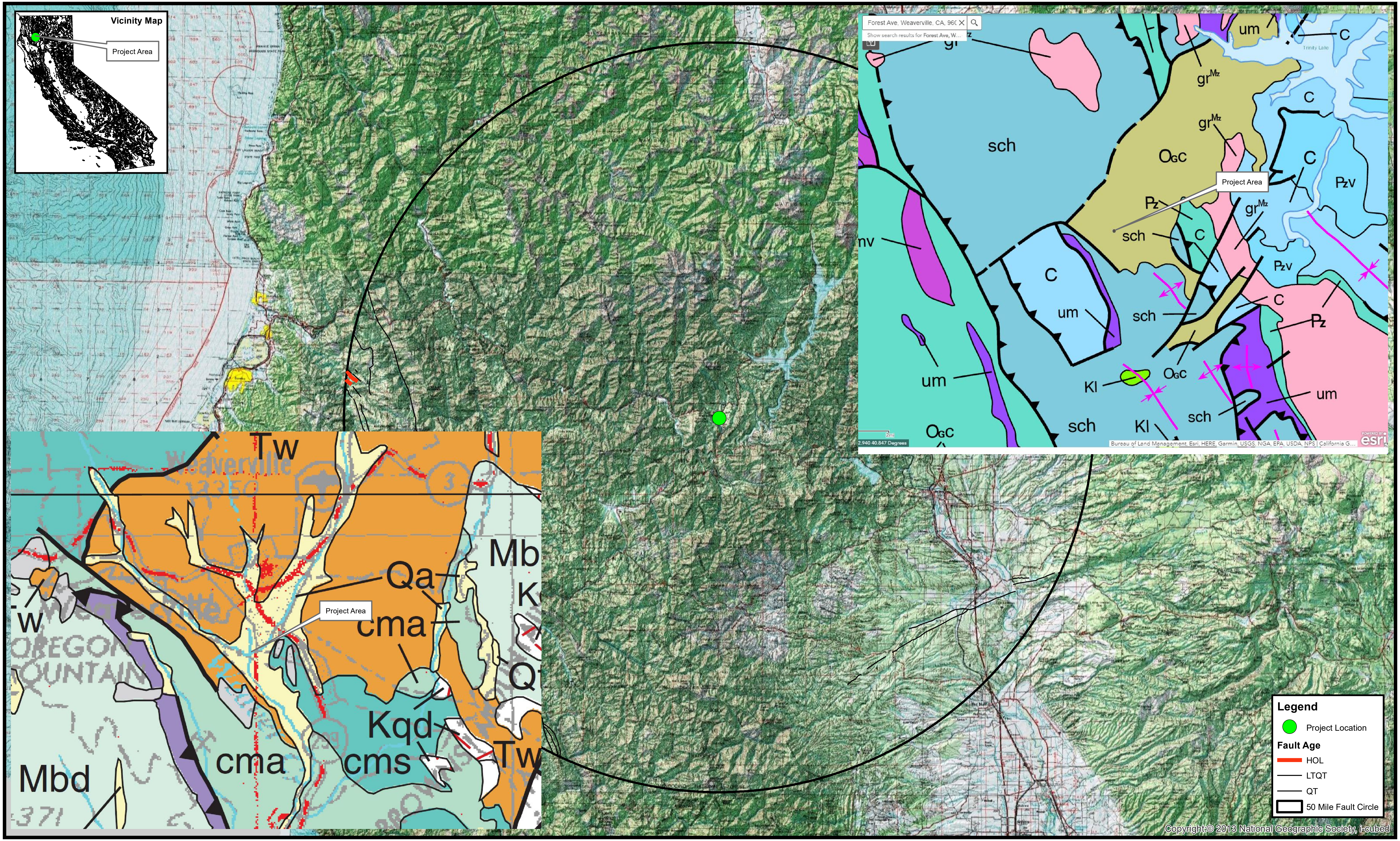
Attachment A



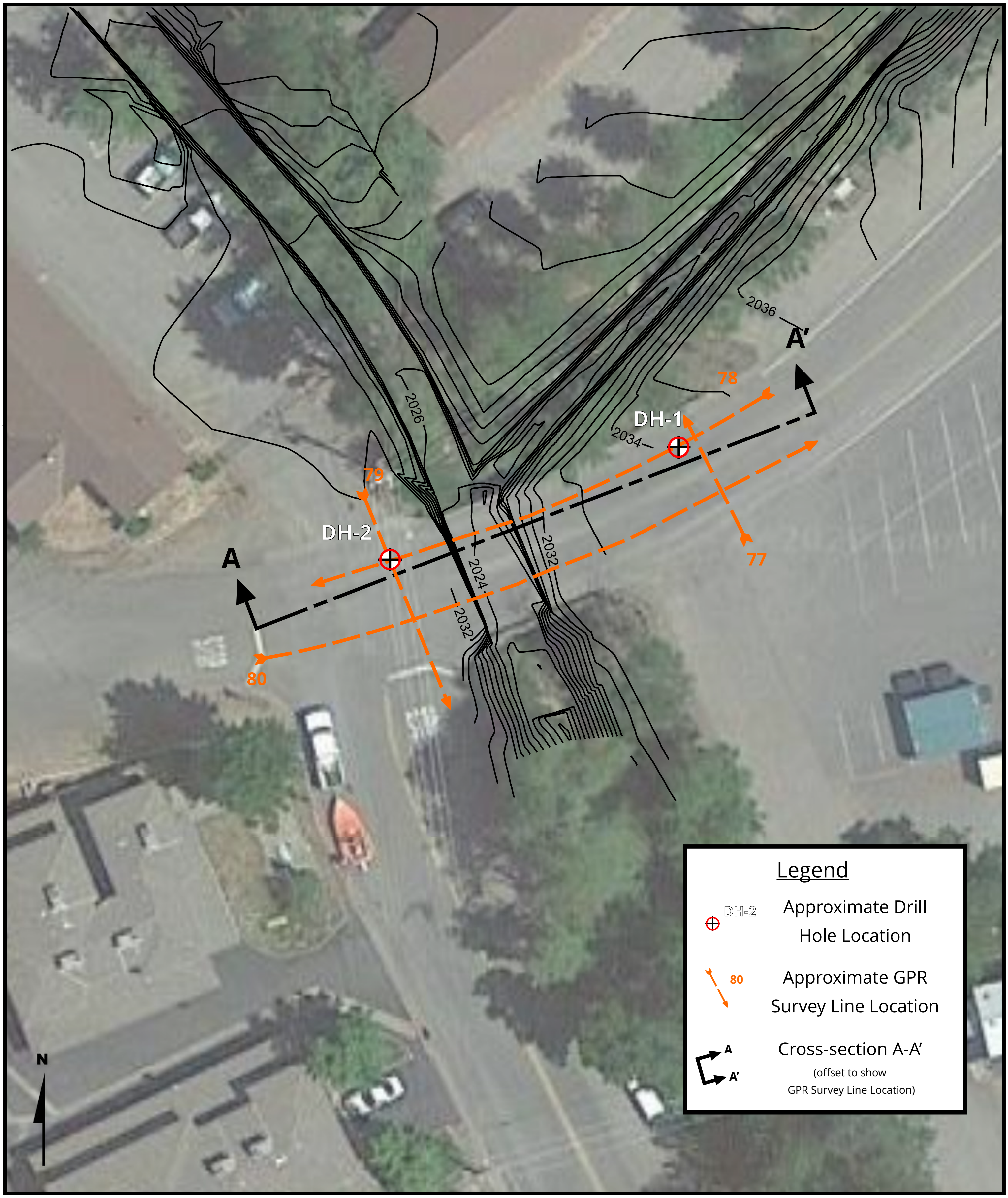
Prepared By:

GeoServ Inc
Engineering Geologists
Rock - Soil - Water

PREPARED FOR: 5Cs
PROJECT: Forest Ave. and Lee Fong Park Geotechnical Investigation
FIGURE 1. Project location, geology, and fault map

N
0 10 20 Miles
Coordinate System: NAD 1983 StatePlane California I FIPS 0401 Feet
Projection: Lambert Conformal Conic
Datum: North American 1983

Date: 5/28/2020



GeoServ
Geoscience Services

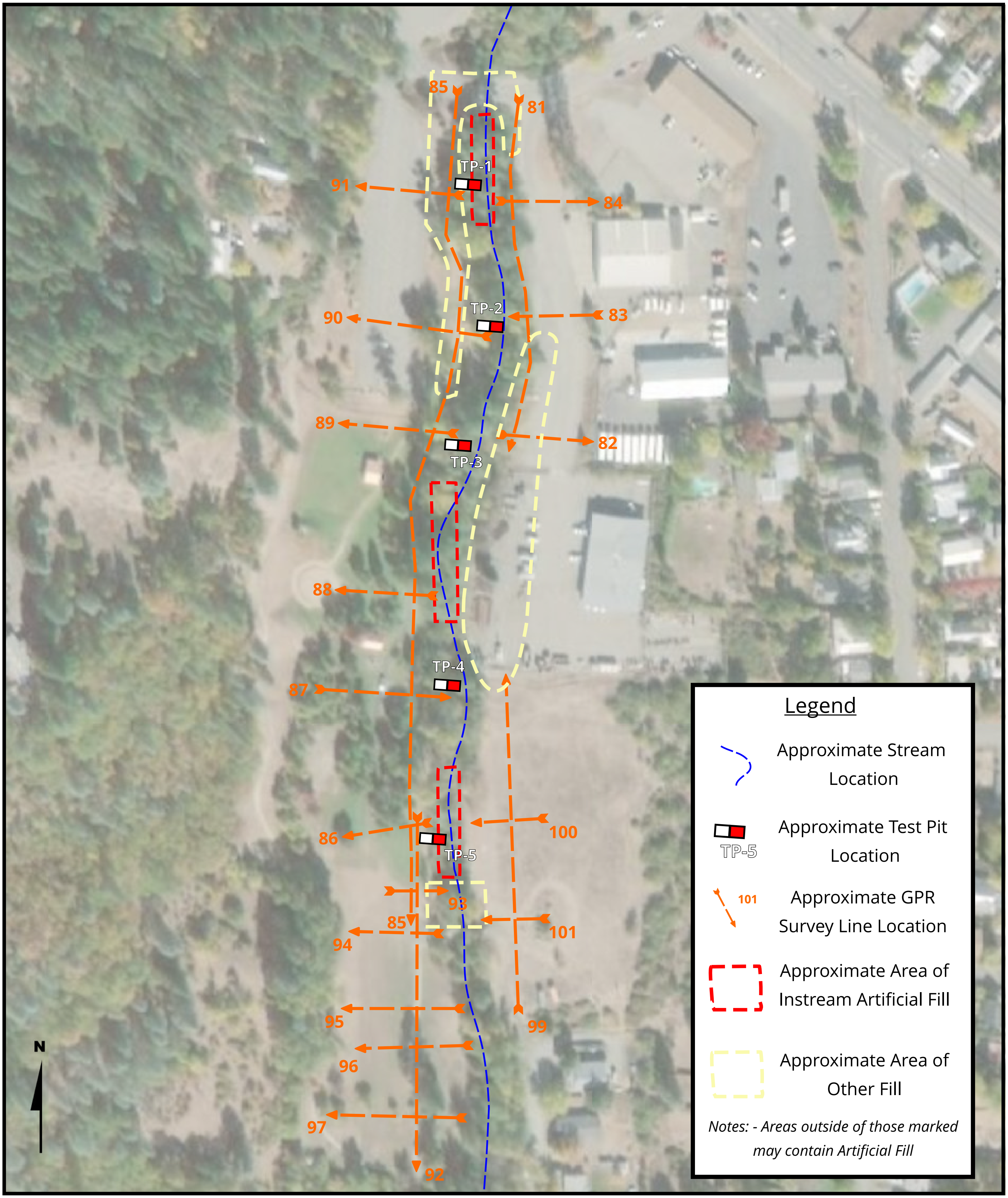
Geotechnical Map

Project: Forest Ave and Lee Fong Park Geotechnical Investigation

Client: Five Counties Salmonid Conservation Program

Scale: Undetermined

Figure: 2



GEOSERV
Geoscience Services

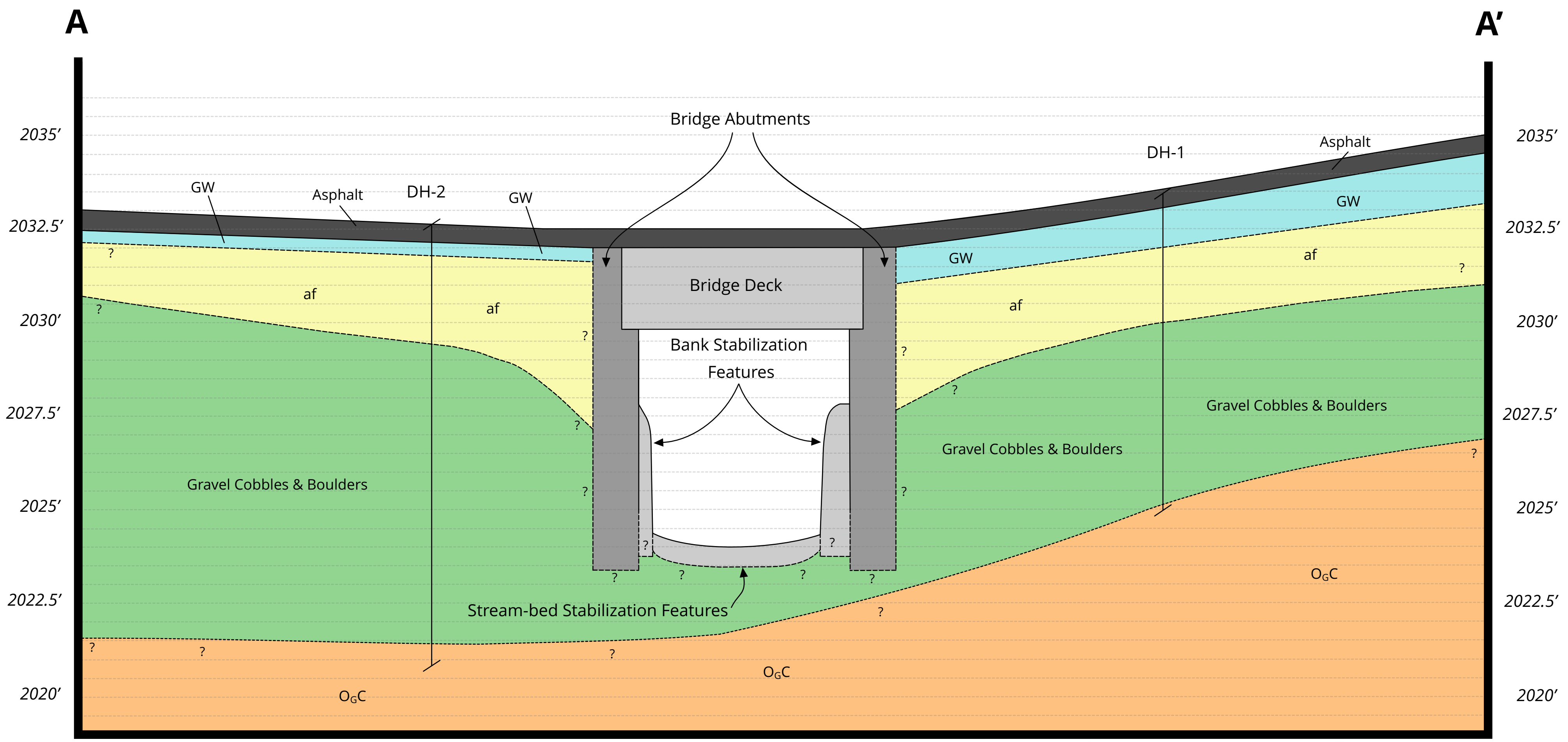
Geotechnical Map

Project: Forest Ave and Lee Fong Park Geotechnical Investigation

Client: Five Counties Salmonid Conservation Program

Scale: Undetermined

Figure: 3



af	Artificial Fill - Sandy Gravel with Cobbles & Boulders
🟩	Gravel Cobbles & Boulders
GW	Aggregate Base - Well Graded Gravel
O_GC	Weaverville Formation
╱ ‒ ‒	Geologic Contact (dashed where inferred)
▽	Ground Water Table (dashed where inferred)

GeoServ
Geoscience Services

Cross-Section A-A'

Project: Forest Ave and Lee Fong Park Geotechnical Investigation

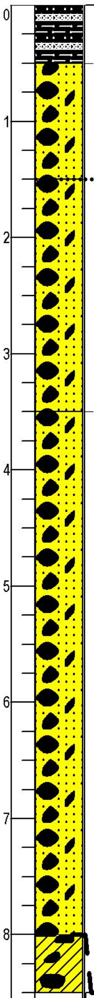
Client: Five Counties Salmonid Conservation Program

Scale: 2.5ft / 20ft

Figure: 4

LOGGED BY **JF & JS**	BEGIN DATE **April 30th, 2020**	COMPLETION DATE **April 30th, 2020**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **40.734031,-122.942233**
DRILLING CONTRACTOR **GeoServ, Inc.**			BOREHOLE LOCATION (Offset, Station, Line) **NA**
DRILLING METHOD **Hollow Stem Auger (6" dia.) / Tri Cone (3" dia.)**			DRILLING RIG **Auger**
SAMPLER TYPE(S) AND SIZES (ID) **CA Split Spoon 2"**			STP HAMMER TYPE **Saftey Hammer**
BOREHOLE BACKFILL AND COMPLETION **Bentonite Chip, 5/1/2020**			GROUNDWATER READINGS — DURING DRILLING **ND** — AFTER DRILLING (DATE) **ND**

HOLE ID	**DH-1**
SURFACE ELEVATION	**~2038.0'**
BOREHOLE DIAMETER	**6" & 3"**
HAMMER EFFICIENCY, ERi	**NA**
TOTAL DEPTH OF BORING	**8.5'**



ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	PLASTIC LIMIT, (%)	PLASTICITY INDEX, (%)	LIQUID LIMIT, (%)	Drilling Method	Casing Depth	Remarks
	0	Asphalt: 6"-6.5" Thick Asphalt												
	1	GW: AGGREGATE BASE - SANDY GRAVEL: FILL-GW; brown; moist; moderately dense; SAND fine to coarse; GRAVEL <1.5" dia. subangular to angular.												8" dia. AC core
	2	GW: ARTIFICIAL FILL - CLAYEY GRAVEL WITH SAND: FILL-GW; brown; moist; moderately dense; CLAY low to medium plasticity; SAND coarse; GRAVEL <1.5" dia. subangular to angular, composed of greenstone.			6/12/14	26	70							
2035	3													
	4	GW: CLAYEY GRAVEL WITH COBBLES AND BOULDERS: GW; brown; moist; moderately dense to very dense; CLAY low to medium plasticity; SAND coarse; GRAVEL <1.5" dia. subangular to angular; COBBLES <8" dia.; BOULDER <18" dia.										6 Hours		6" Auger refusal @ 3.5' bgs 3" Tri-cone/mud 3.5'-8.5' bgs
	5													
	6													Harder drilling 3.5'-6.5' bgs
	7													
2030	8	GC: CLAYEY GRAVEL WITH COBBLES AND BOULDERS: QLS-GC; brown; moist; very dense; CLAY medium plasticity; SAND coarse; GRAVEL <1.5" dia. subangular to angular; COBBLES <8" dia.; BOULDER <18" dia.												Weaverville Formation @ 8.0'
	9													
	10													

REPORT TITLE **Forest Ave Geotechnical Investigation**				HOLE ID **DH-1**
DIST.	SITE NAME	SITE ID#:	STRUCTURE TYPE:	EA
SITE ADDRESS **Forest Ave Bridge At Weaver Creek, Weaverville, CA 96093**				
BRIDGE NUMBER	PREPARED BY **KF**		DATE **5/19/20**	SHEET **1**

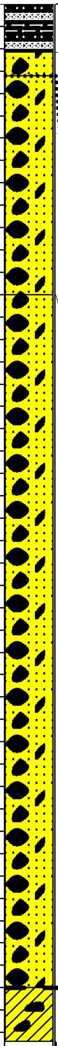
LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	**May 1st, 2020**	**May 1st, 2020**	**40.733932,-122.942531**	**DH-2**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	**NA**	**~2037.0'**

DRILLING METHOD	DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger (6" dia.) / Tri Cone (3" dia.)	**Auger**	**6" & 3"**

SAMPLER TYPE(S) AND SIZES (ID)	STP HAMMER TYPE	HAMMER EFFICIENCY, ERi
CA Split Spoon 2"	**Saftey Hammer**	**NA**

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, 5/1/2020		**ND**	**ND**	**11.6'**



ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1.60	PLASTIC LIMIT, (%)	PLASTICITY INDEX, (%)	LIQUID LIMIT, (%)	Drilling Method	Casing Depth	Remarks
	0	Asphalt: 8.0" Thick Asphalt										■	■	
	1	GW: AGGREGATE BASE - SANDY GRAVEL: FILL-GW; brown; moist; moderately dense; SAND fine to coarse; GRAVEL <1.5" dia. subangular to angular.												8" dia. AC core Harder drilling @ 1.0'-2.5' bgs-Boulder
2035	2	GW: ARTIFICIAL FILL - SANDY GRAVEL WITH COBBLES AND BOULDERS: FILL-GW; brown; moist; moderately dense; CLAY low to medium plasticity; SAND coarse; GRAVEL <1.5" dia. subangular to angular, COBBLE <8" dia. subangular to angular; BOULDER <18" dia.	▦											Bulk sample taken @ 2.0-3.0' bgs
	3	GW: CLAYEY GRAVEL WITH COBBLES AND BOULDERS: GW; brown; moist; moderately dense to very dense; CLAY low to medium plasticity; SAND coarse; GRAVEL <1.5" dia. subangular to angular; COBBLES <8" dia.; BOULDER <24" dia.												Harder drilling @ 3.25' bgs
	4													
	5													3" Tri-cone/mud 0.54'-11.6' bgs
	6											7 Hours		
2030	7													
	8													
	9													Harder drilling @ 8.0'-10.0' bgs-Boulder
	10													
	11	GC: CLAYEY GRAVEL WITH COBBLES AND BOULDERS: QLS-GC; brown; moist; very dense; CLAY medium plasticity; SAND coarse; GRAVEL <1.5" dia. subangular to angular; COBBLES <8" dia.; BOULDER <18" dia.												Harder drilling @ 11' bgs-Weaverville formation
2025	12													Refusal @ 11.6' bgs
	13													

REPORT TITLE				HOLE ID
Forest Ave Geotechnical Investigation				**DH-2**
DIST.	SITE NAME	SITE ID#:	STRUCTURE TYPE:	EA

SITE ADDRESS
Forest Ave Bridge At Weaver Creek, Weaverville, CA 96093

BRIDGE NUMBER	PREPARED BY	DATE	SHEET
	KF	**5/19/20**	**2**

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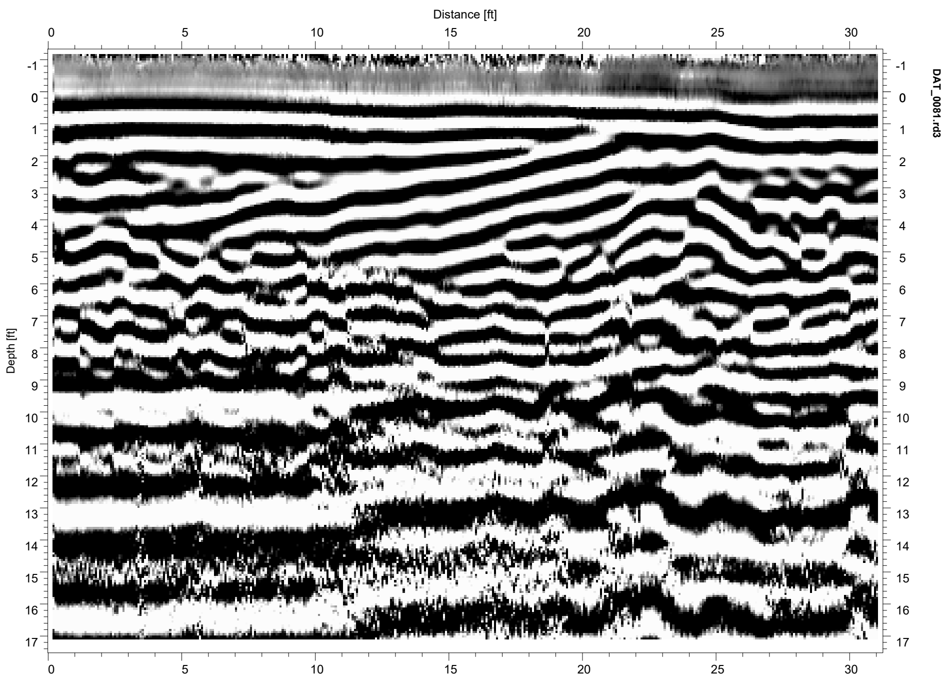

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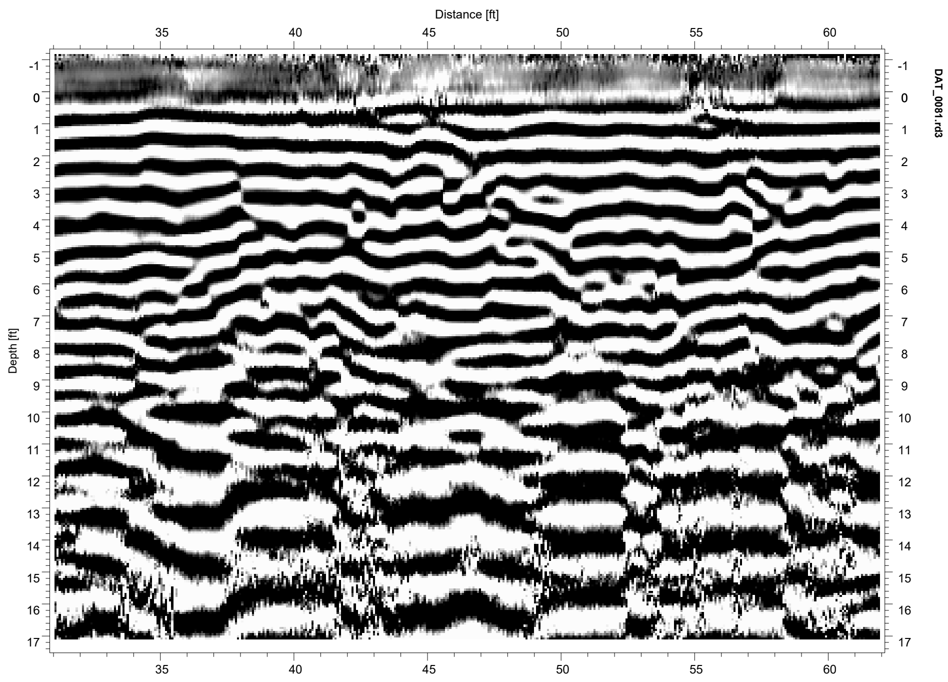

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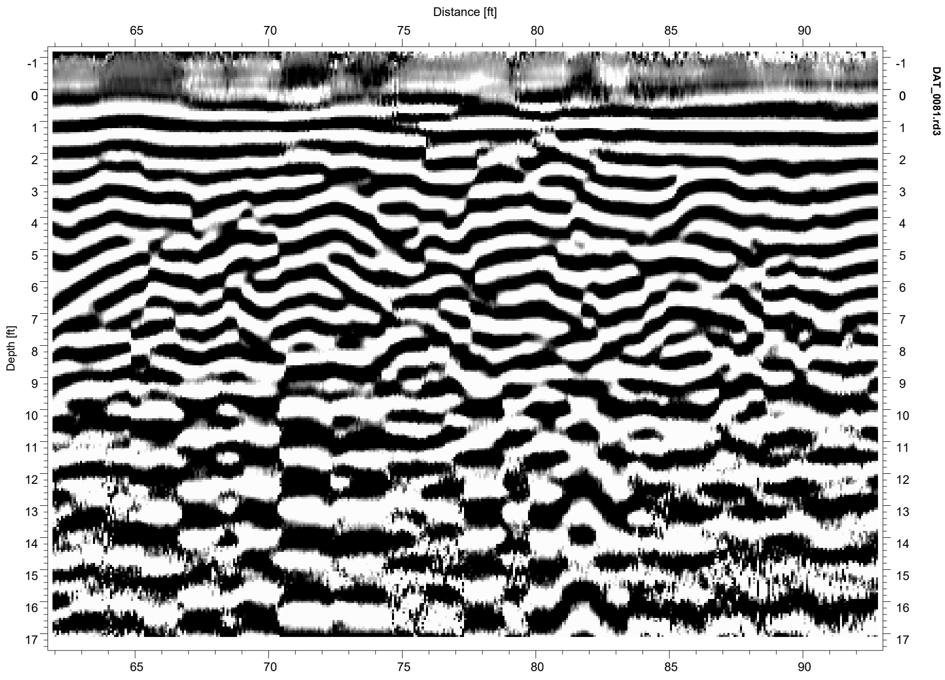

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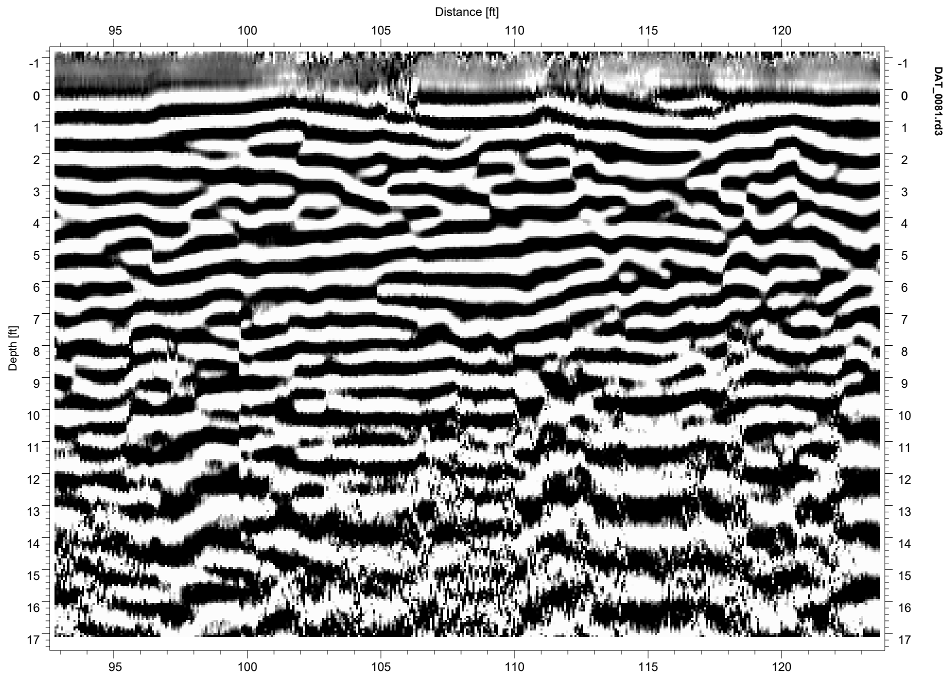

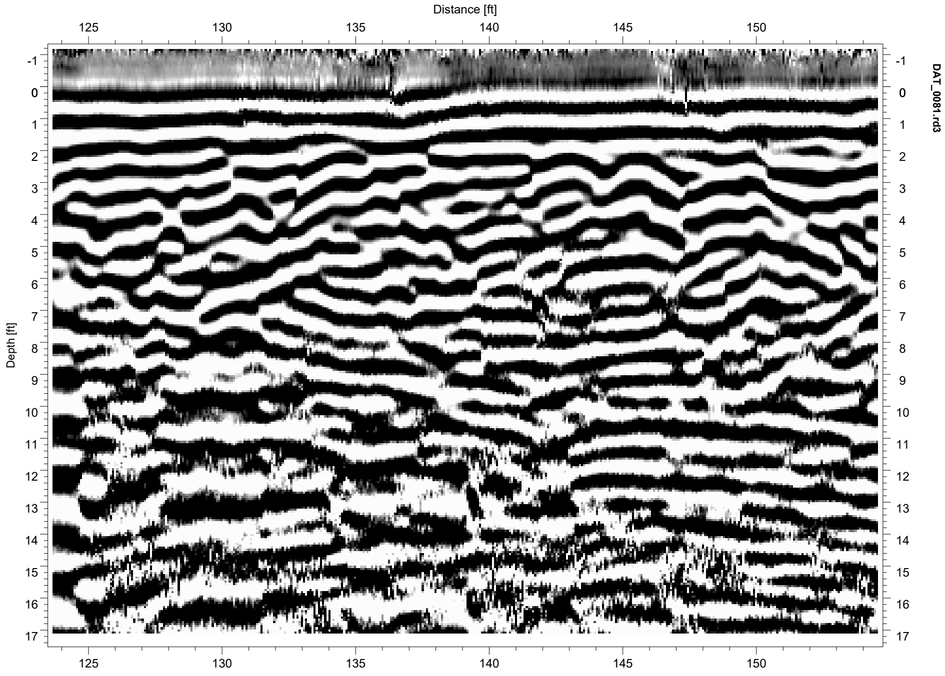
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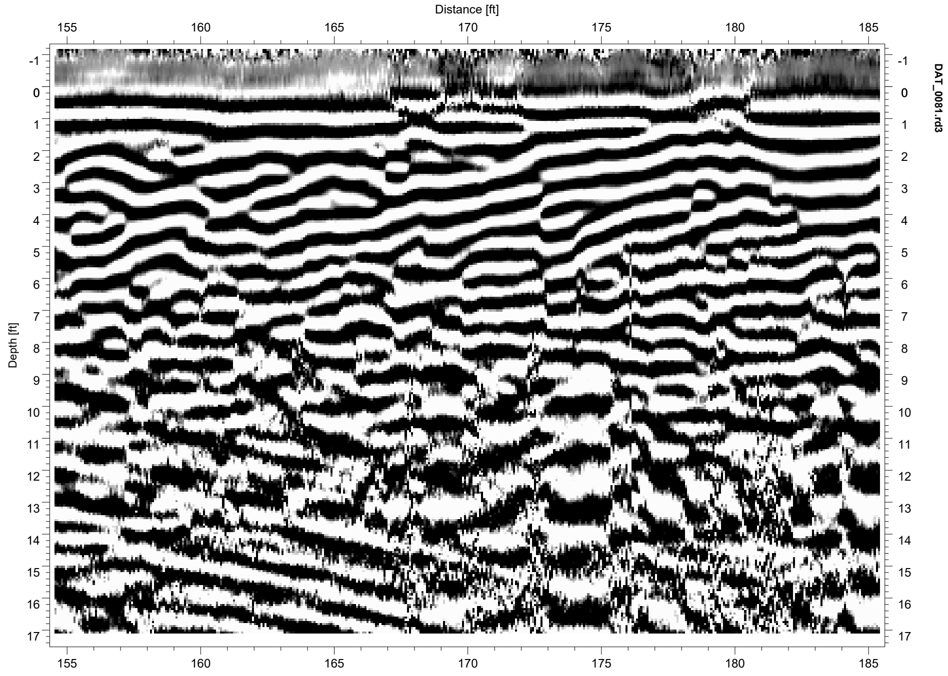
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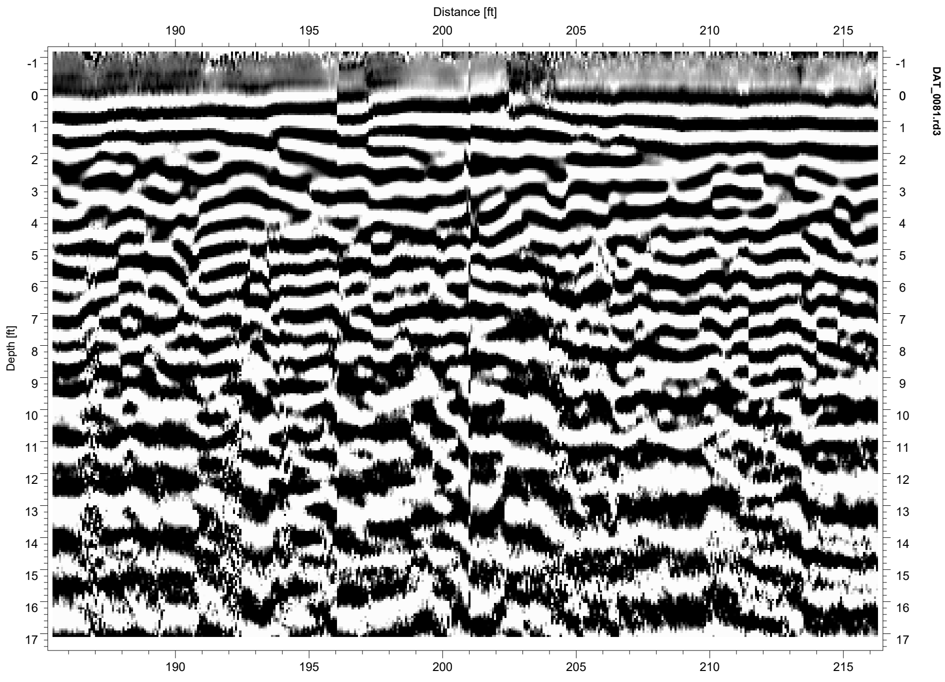
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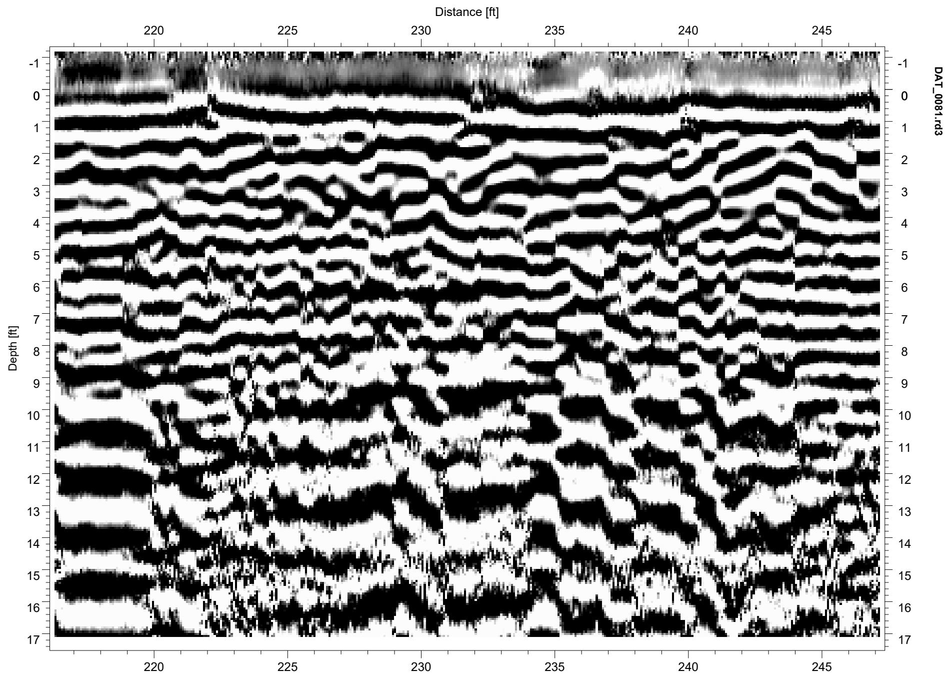
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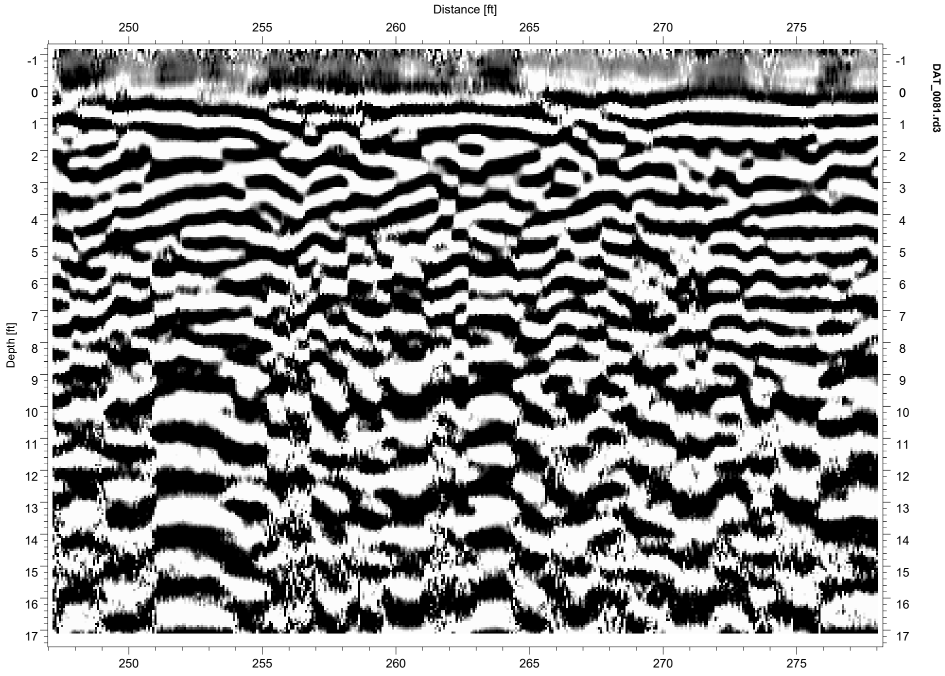
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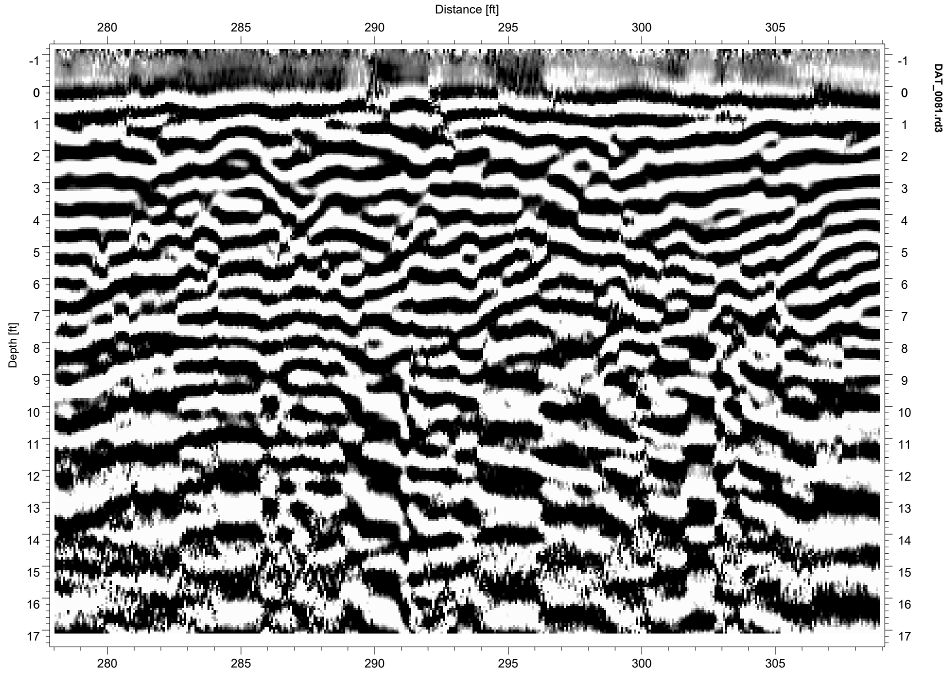
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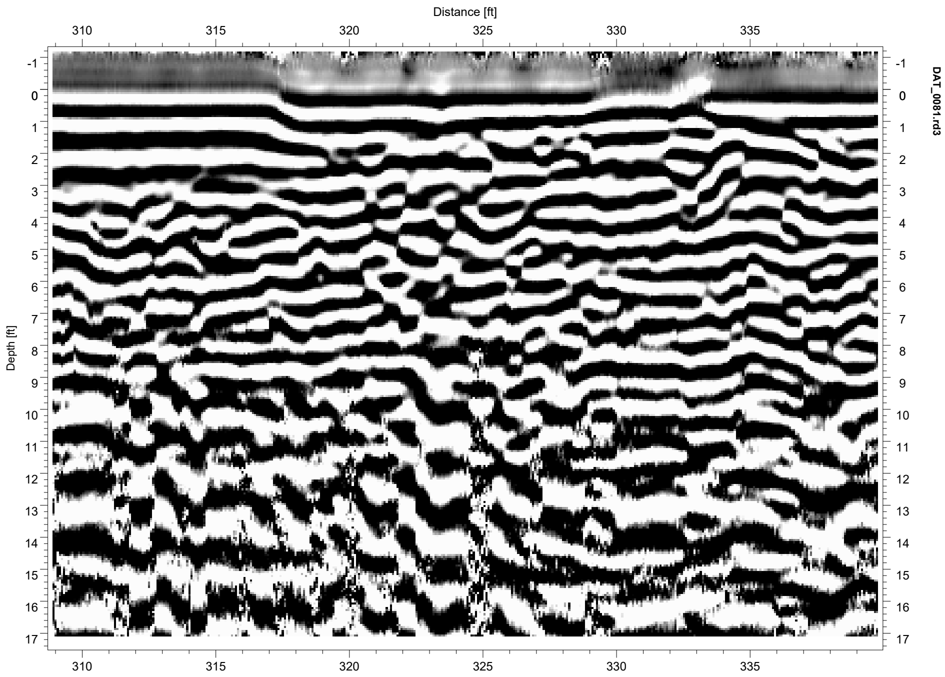
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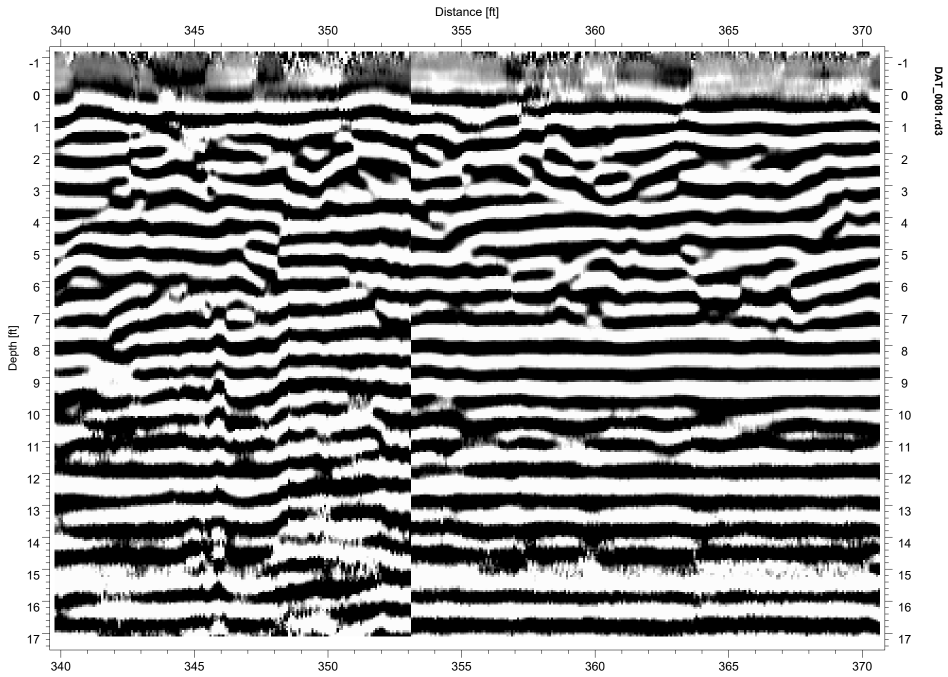
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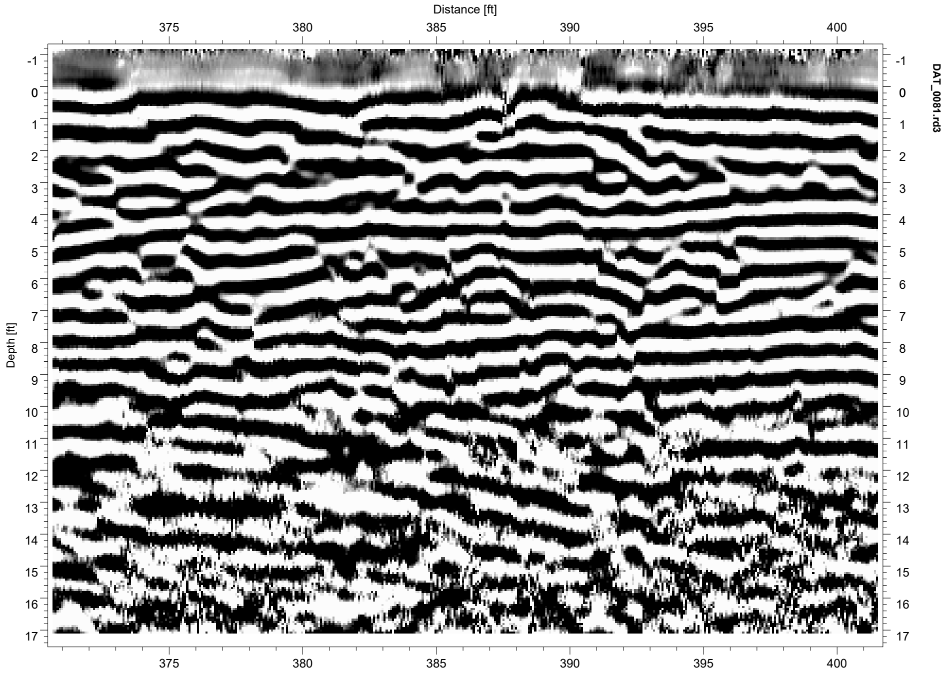

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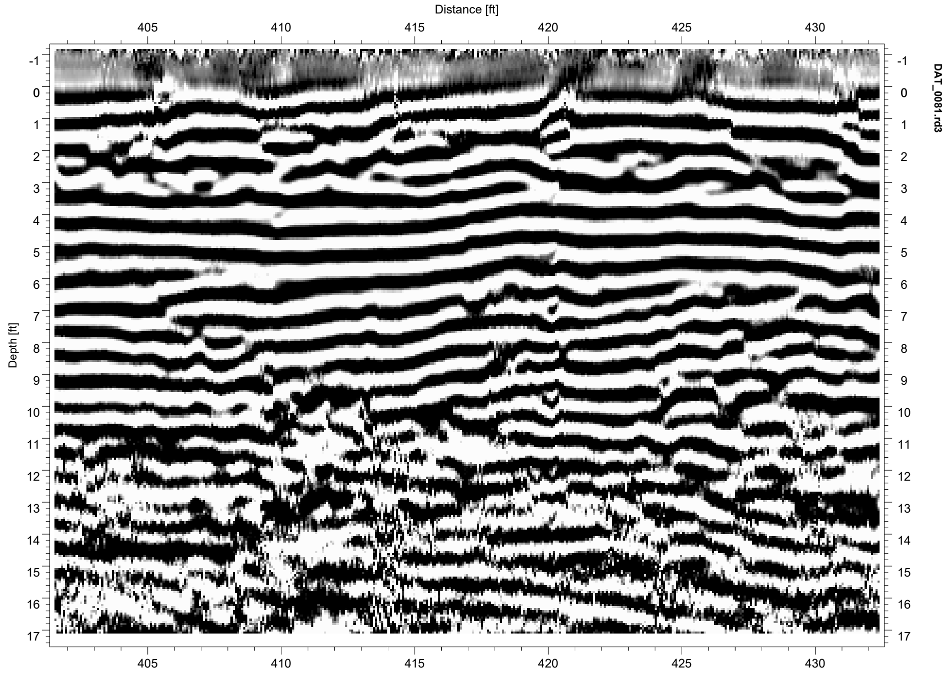

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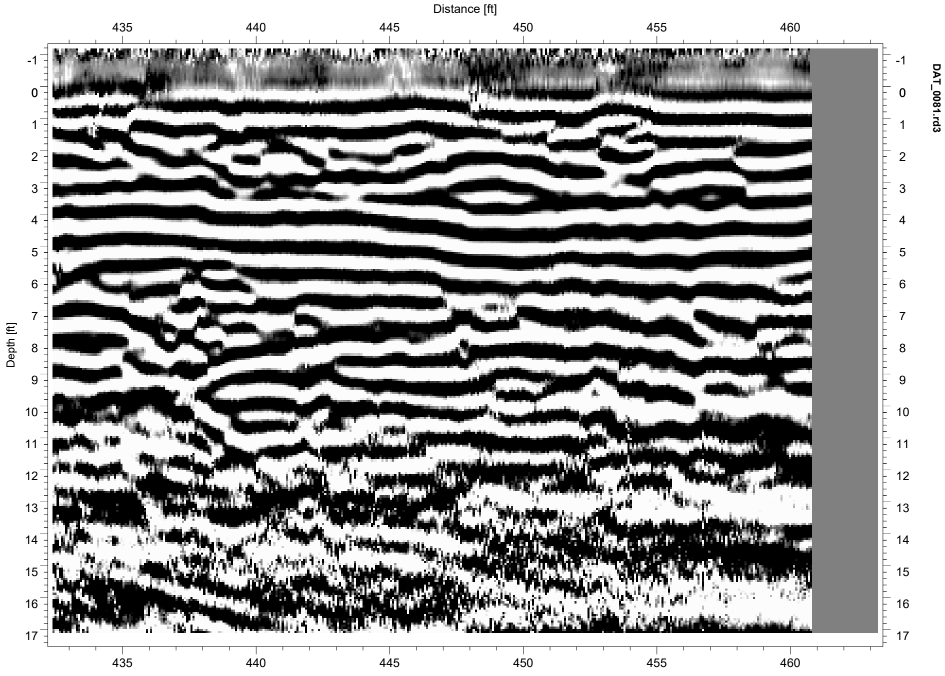

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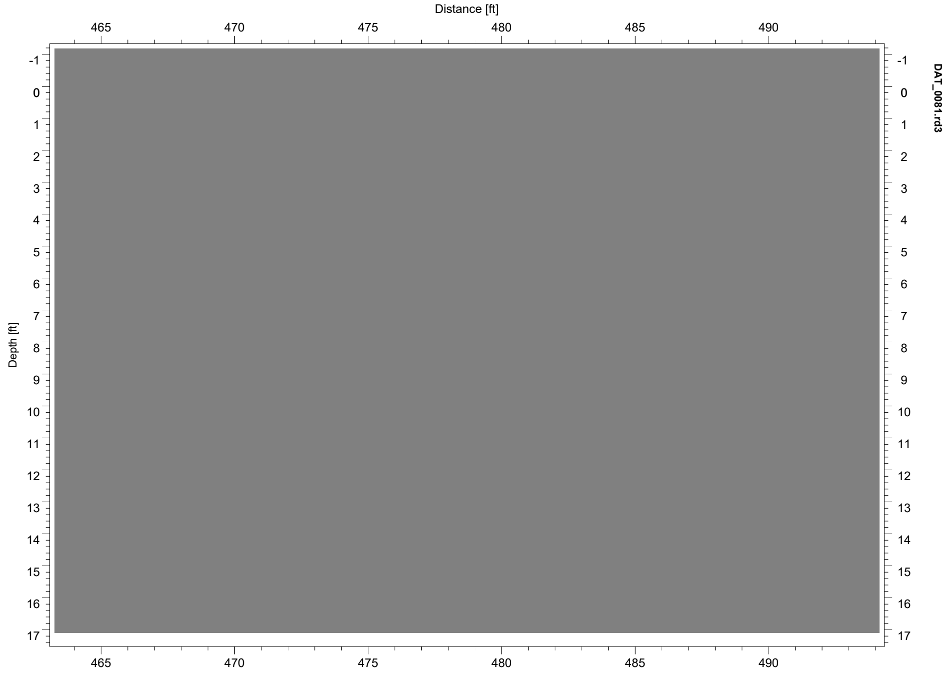

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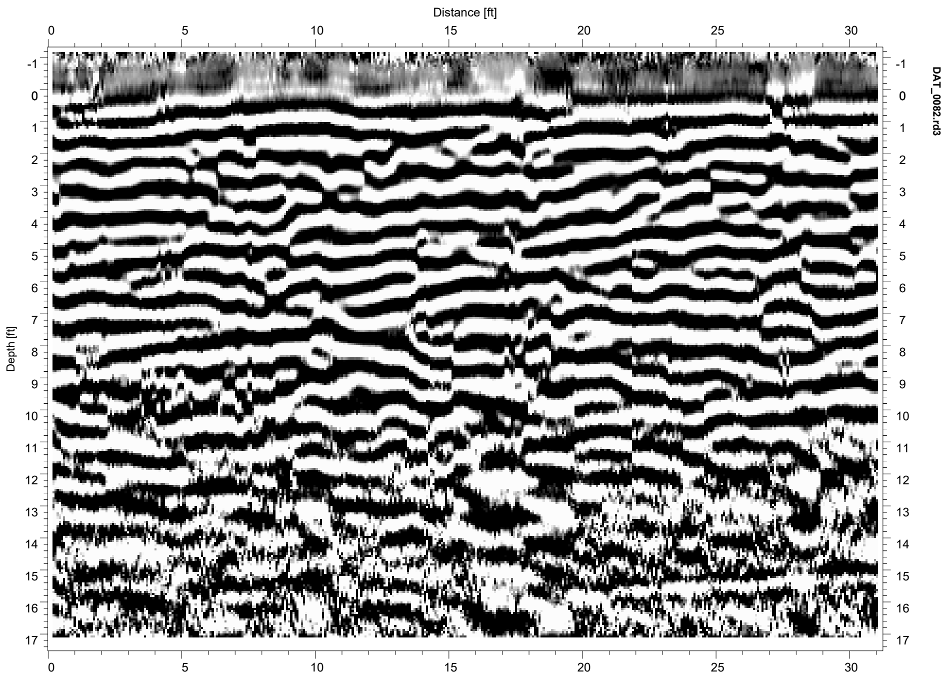

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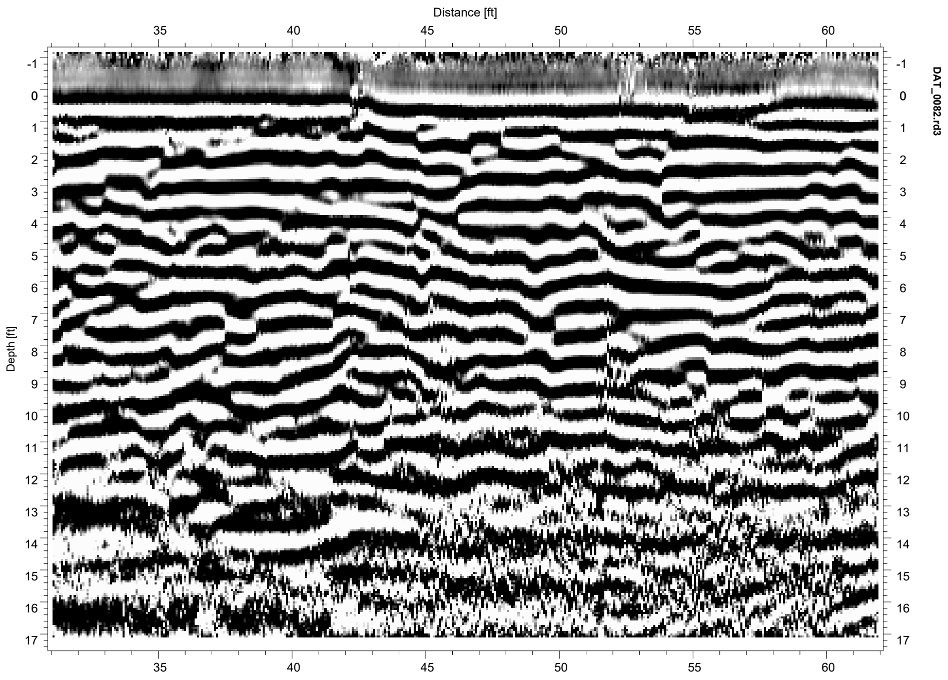

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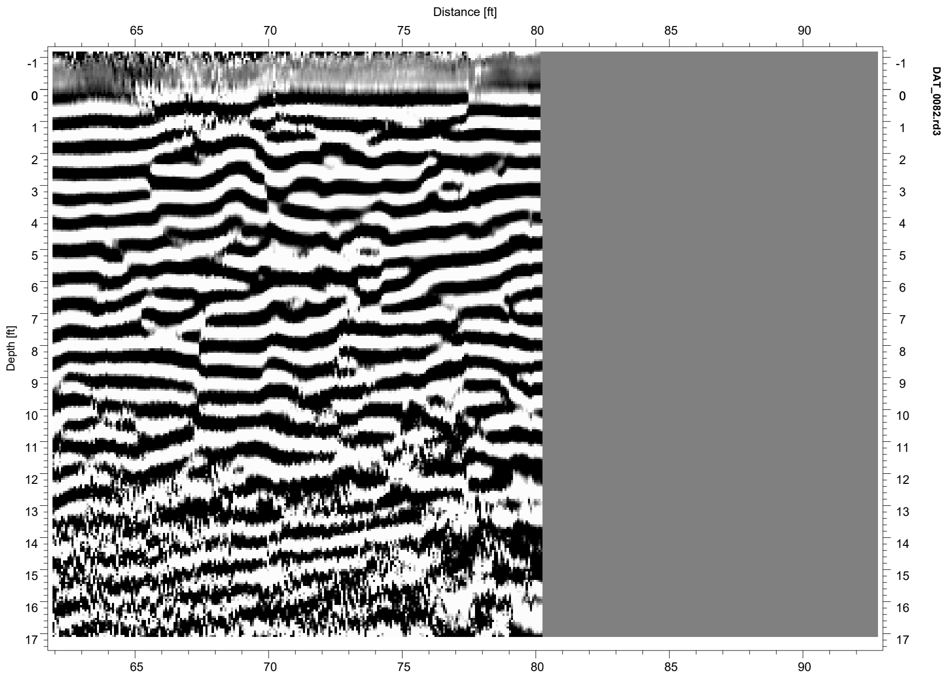

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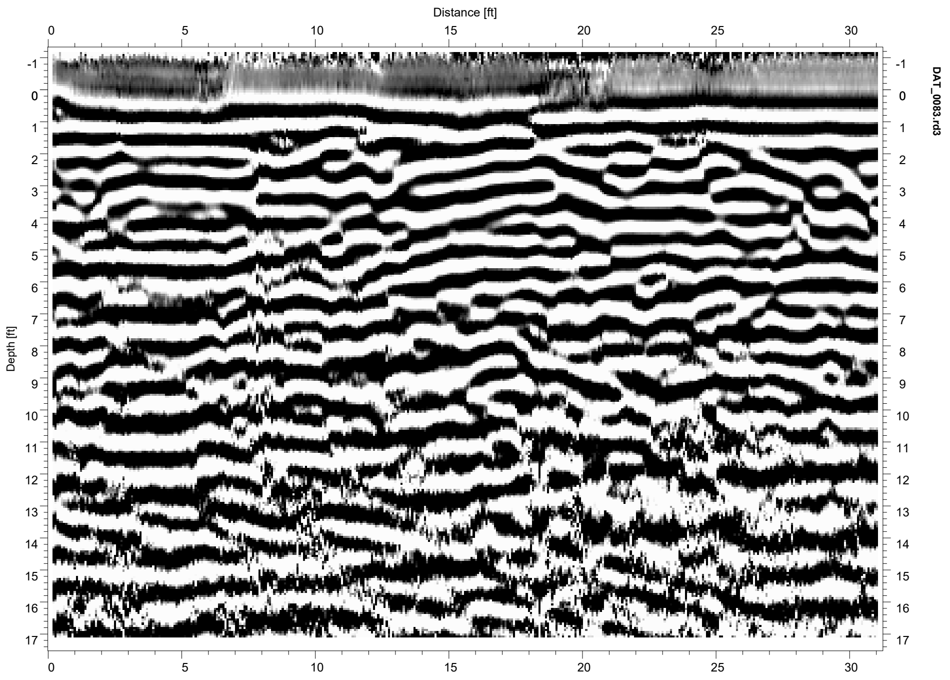

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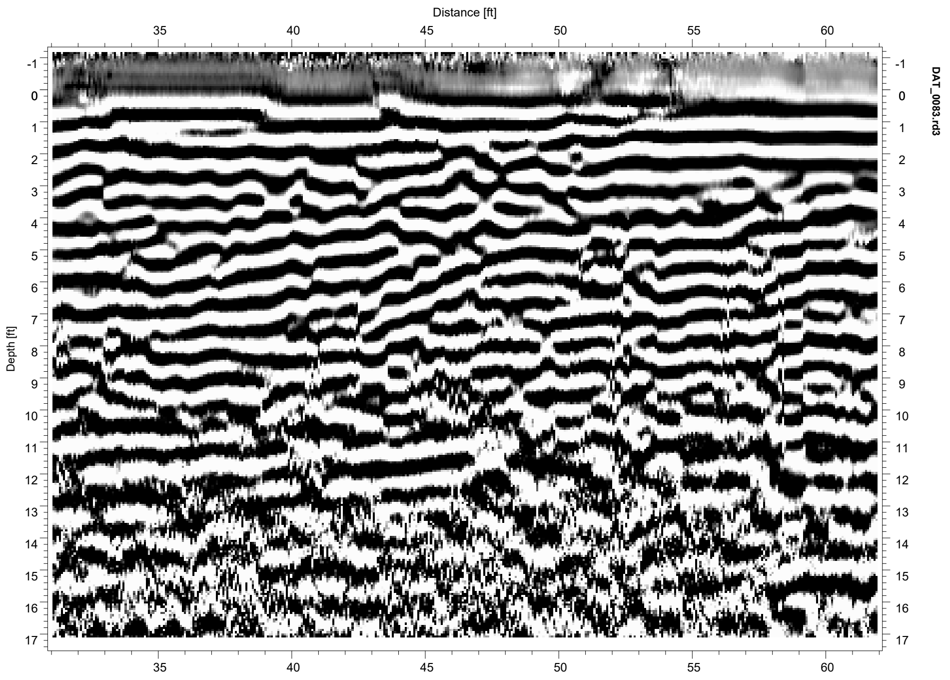

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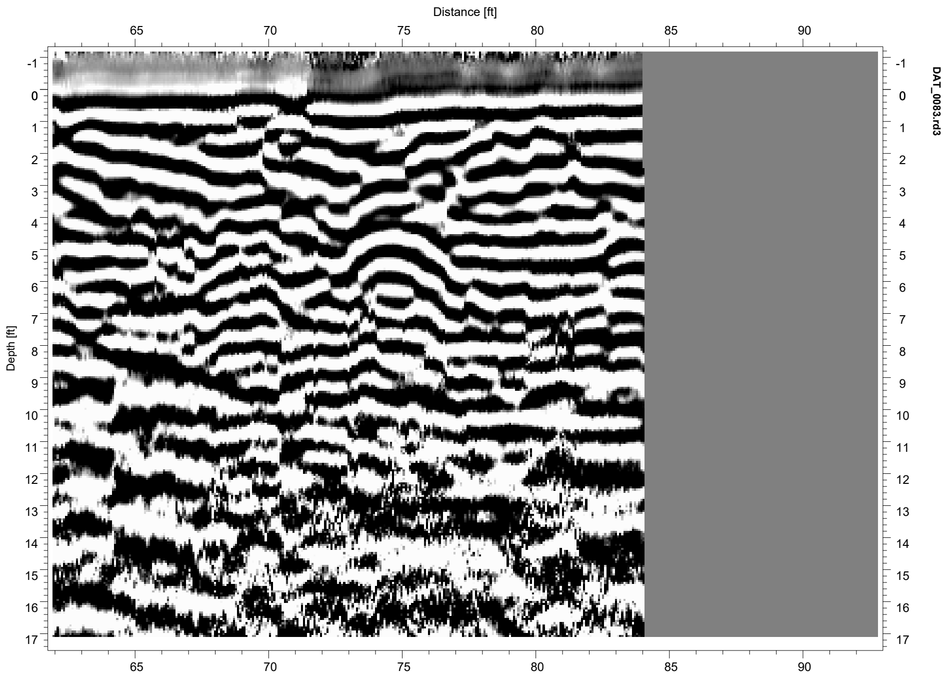

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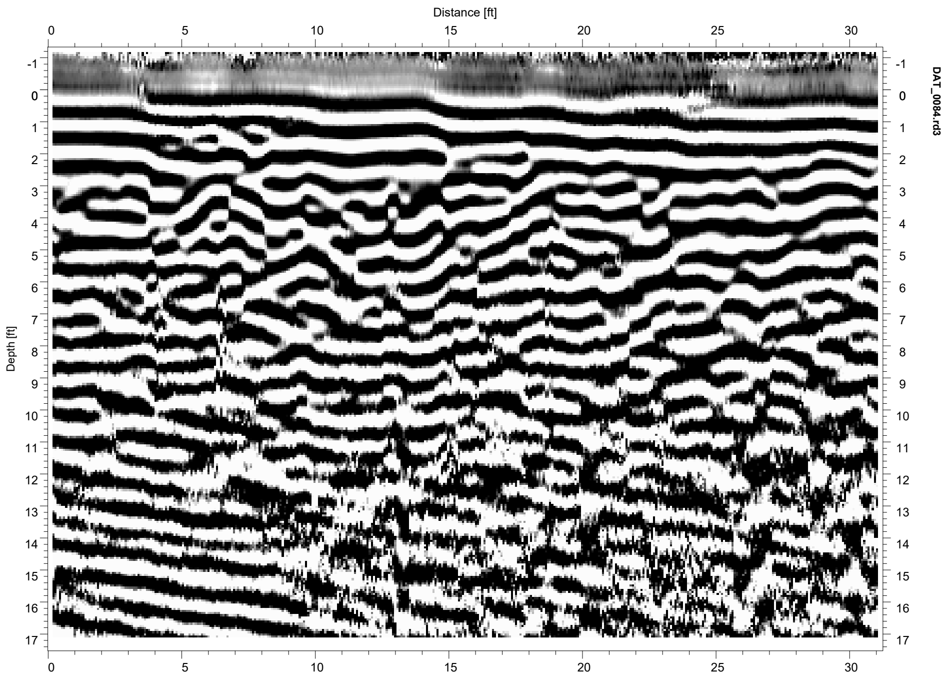

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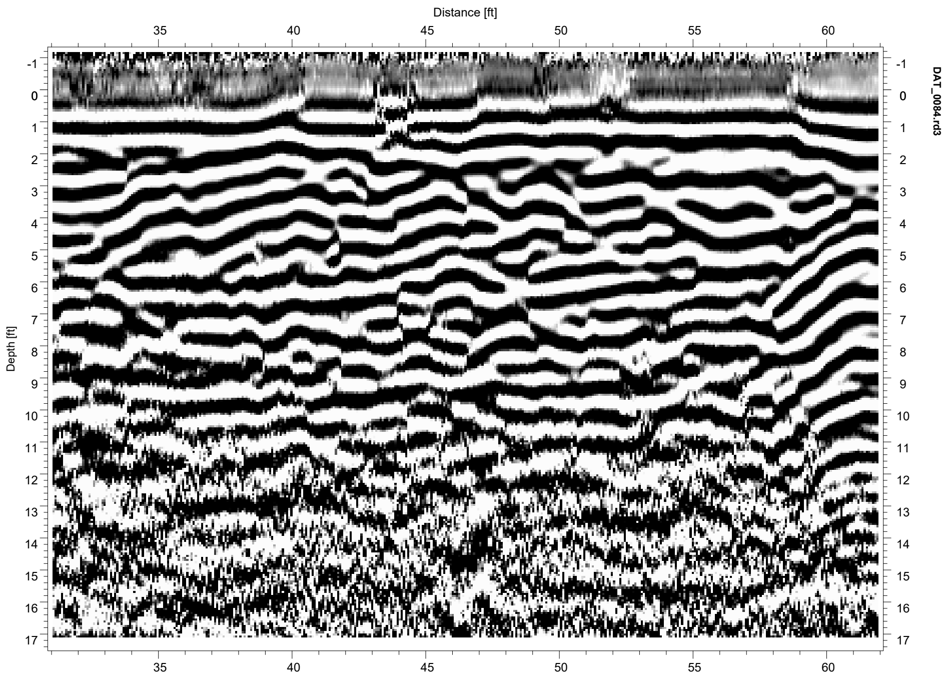

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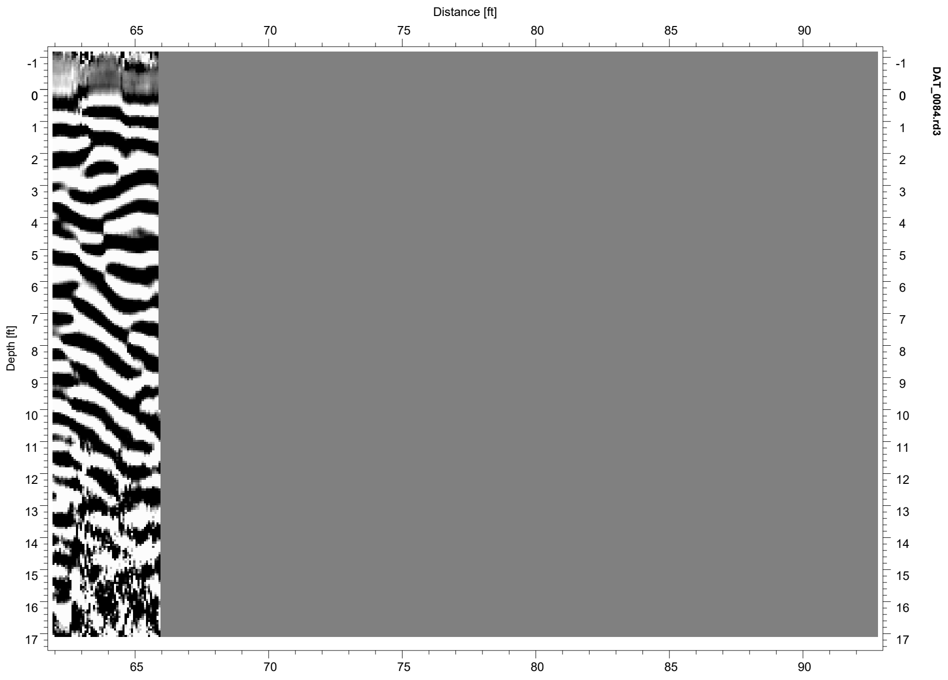

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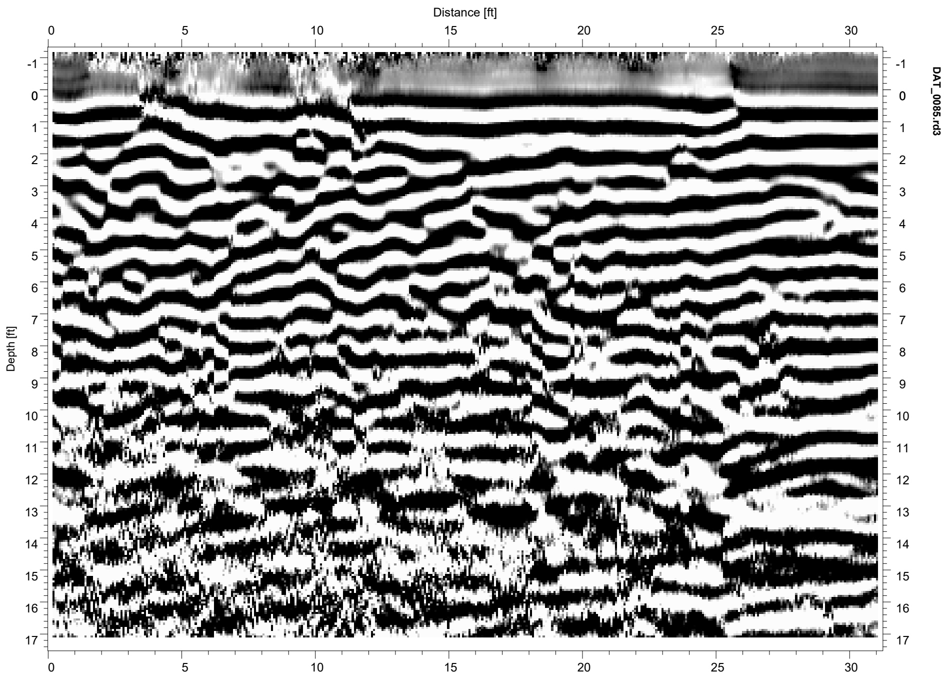

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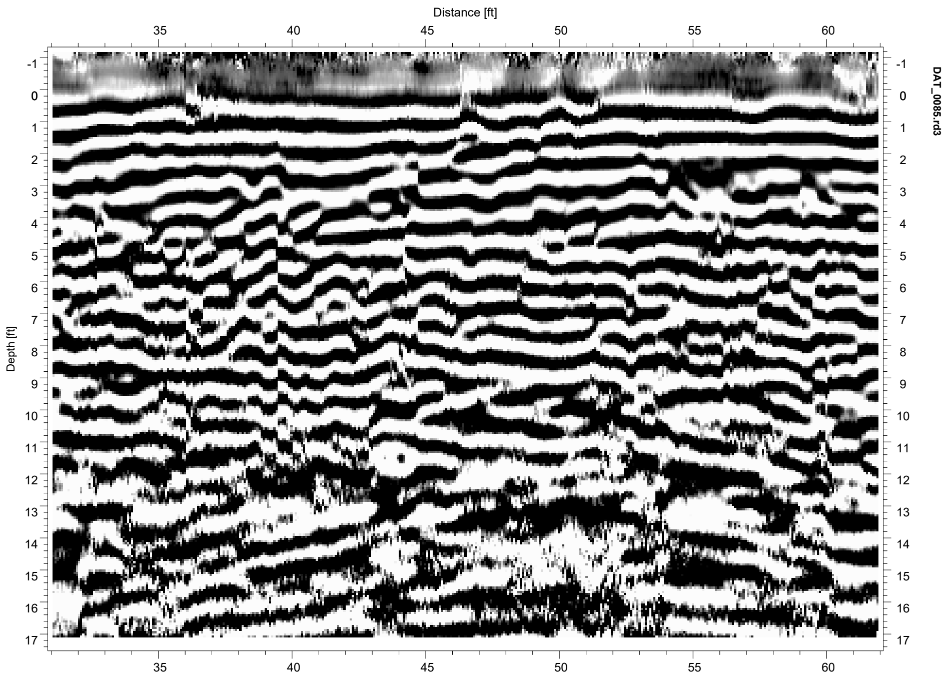

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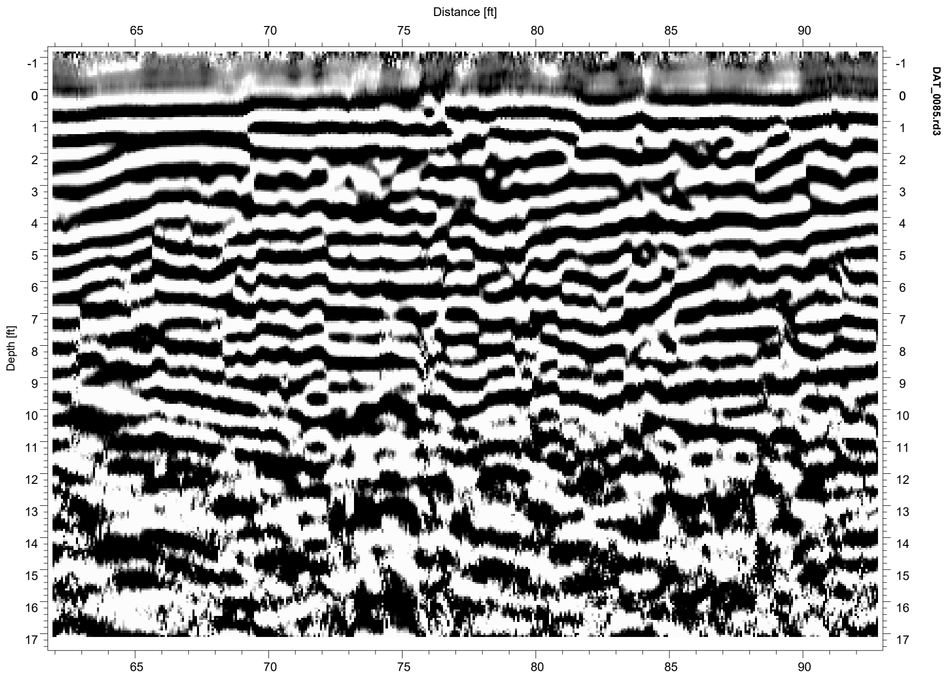

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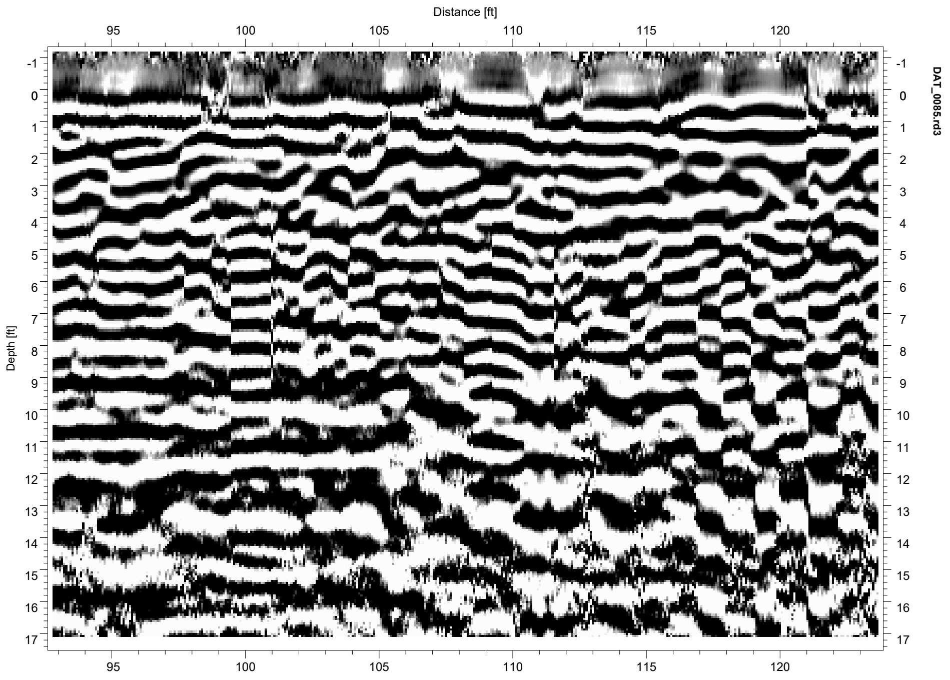

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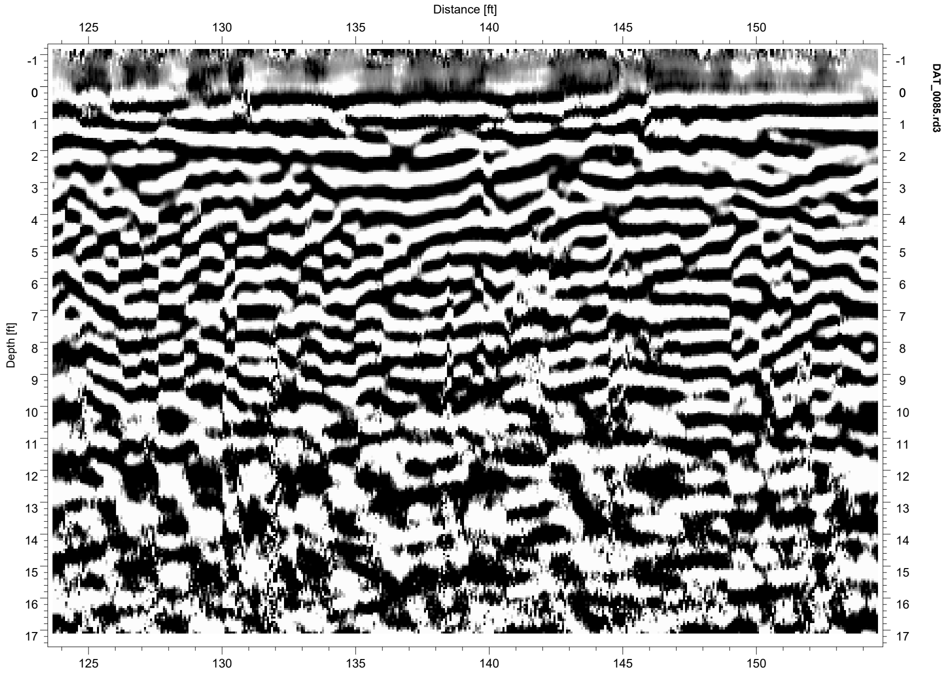

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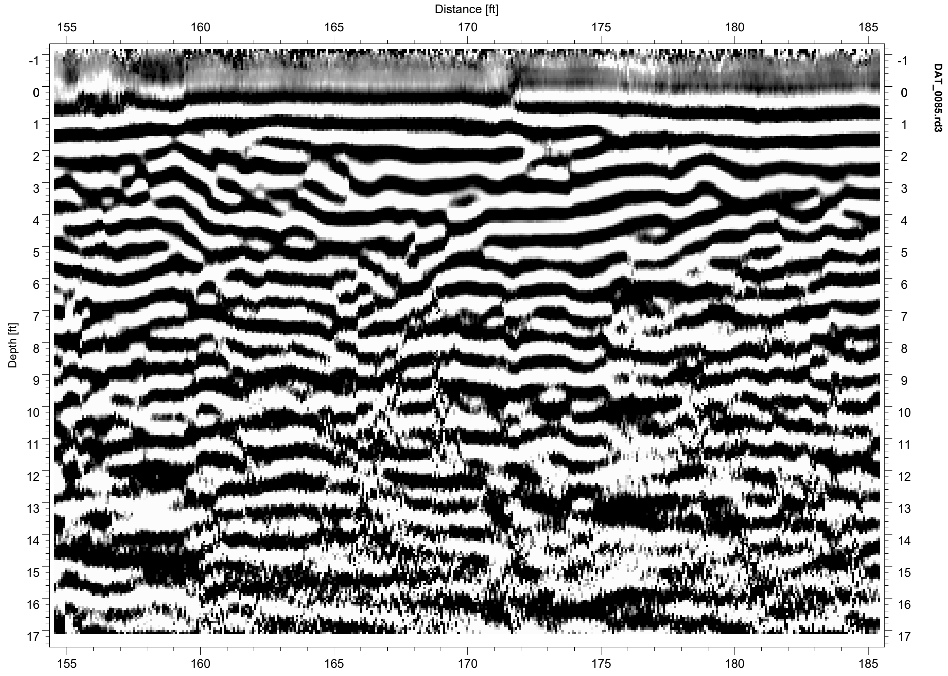

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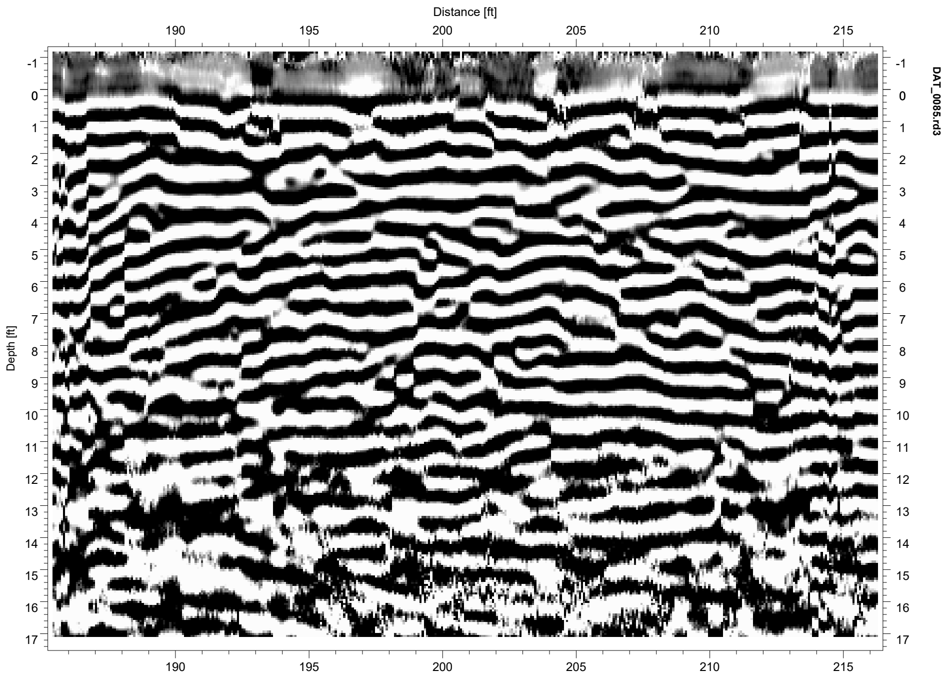

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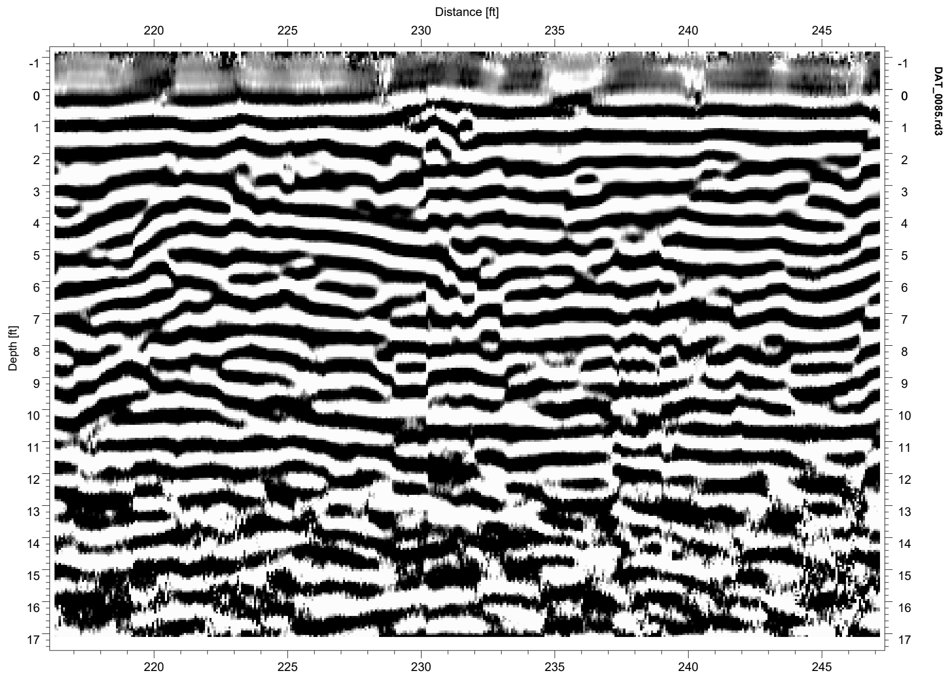

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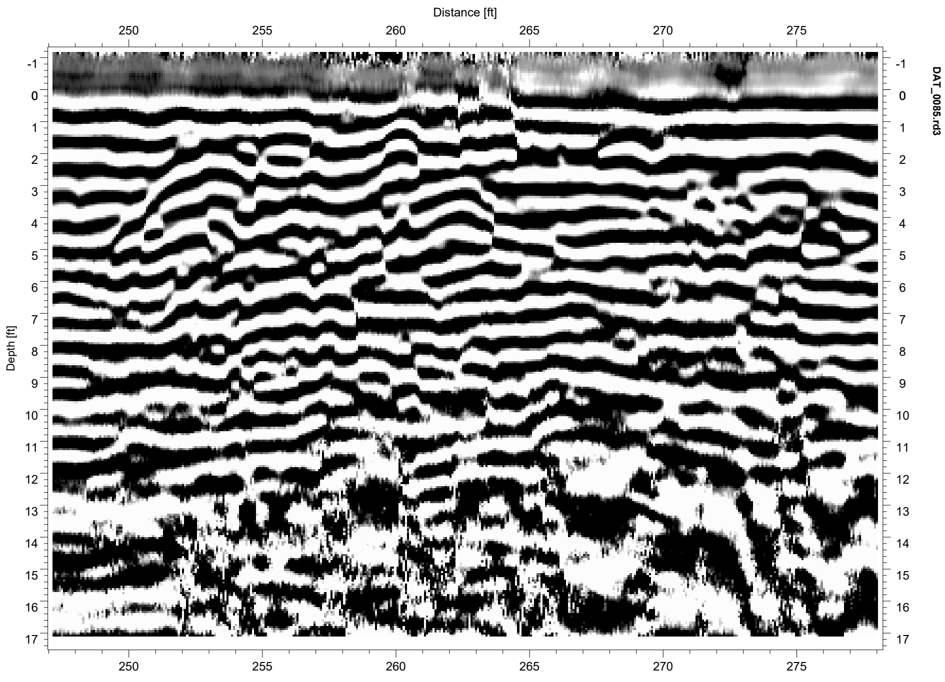

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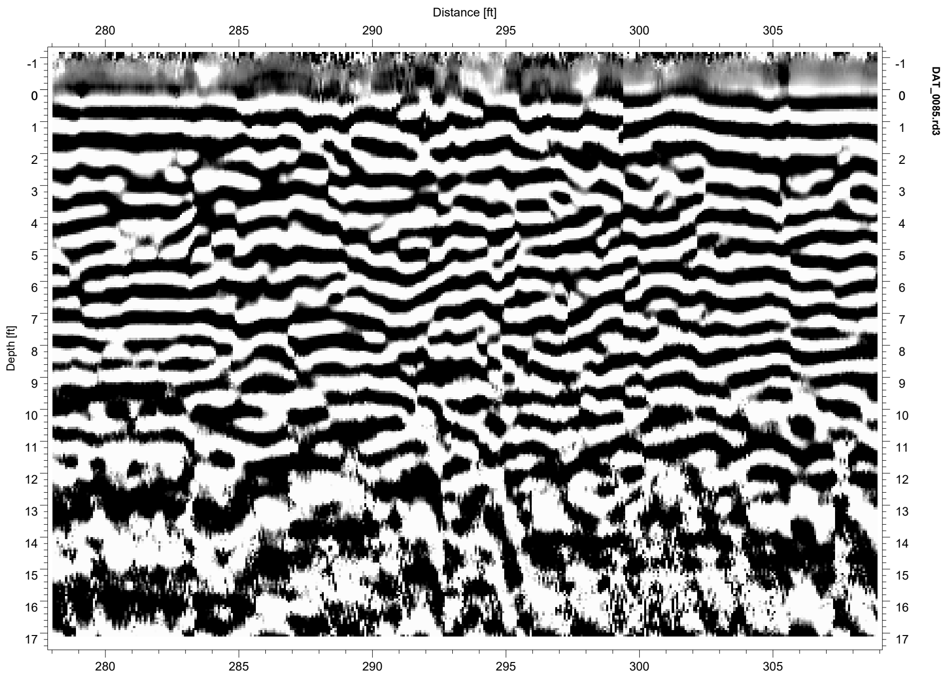

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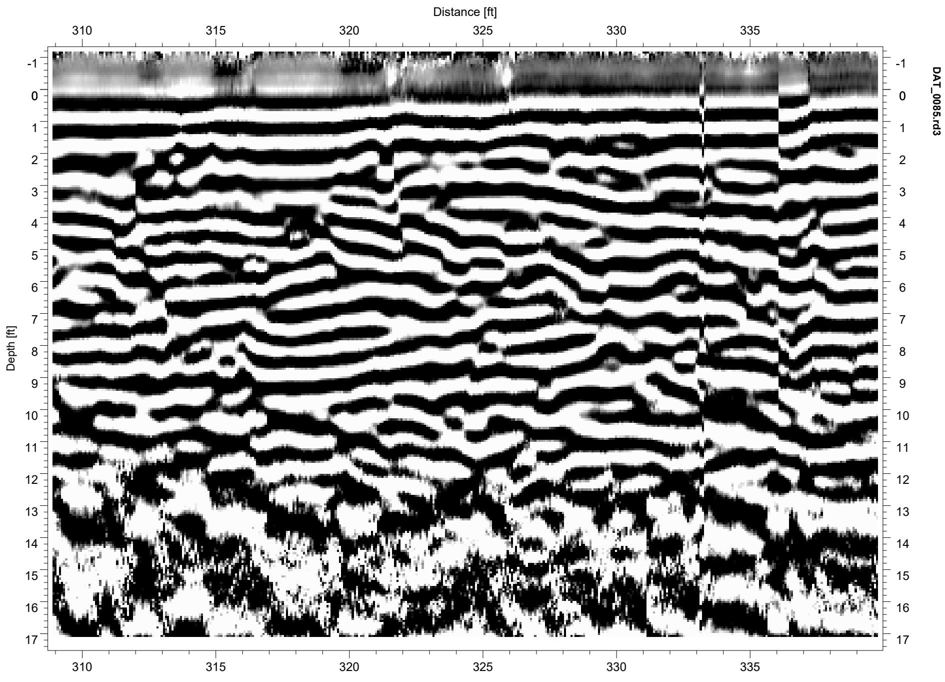

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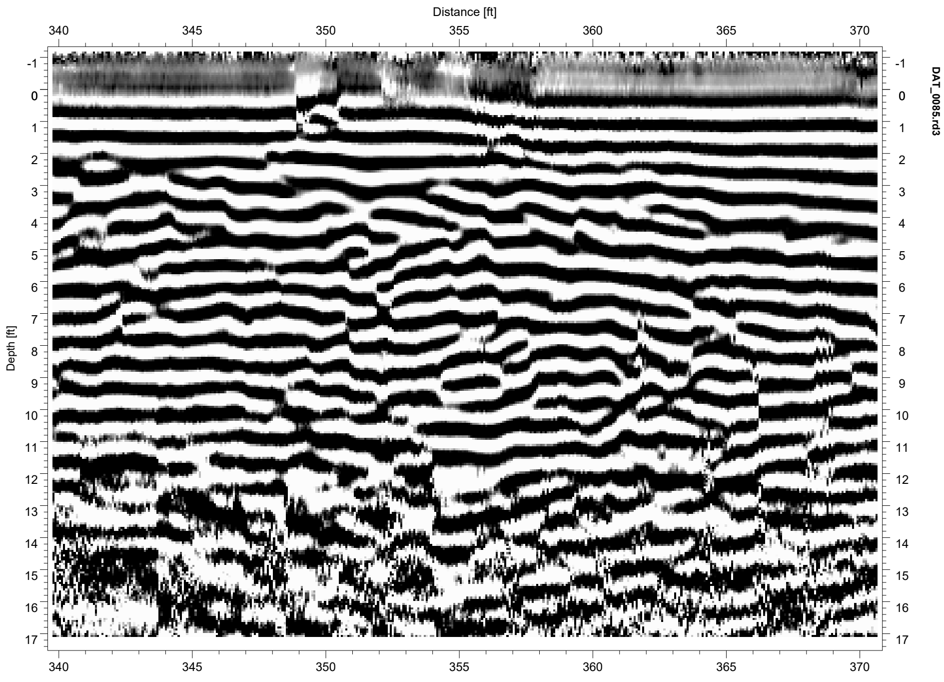

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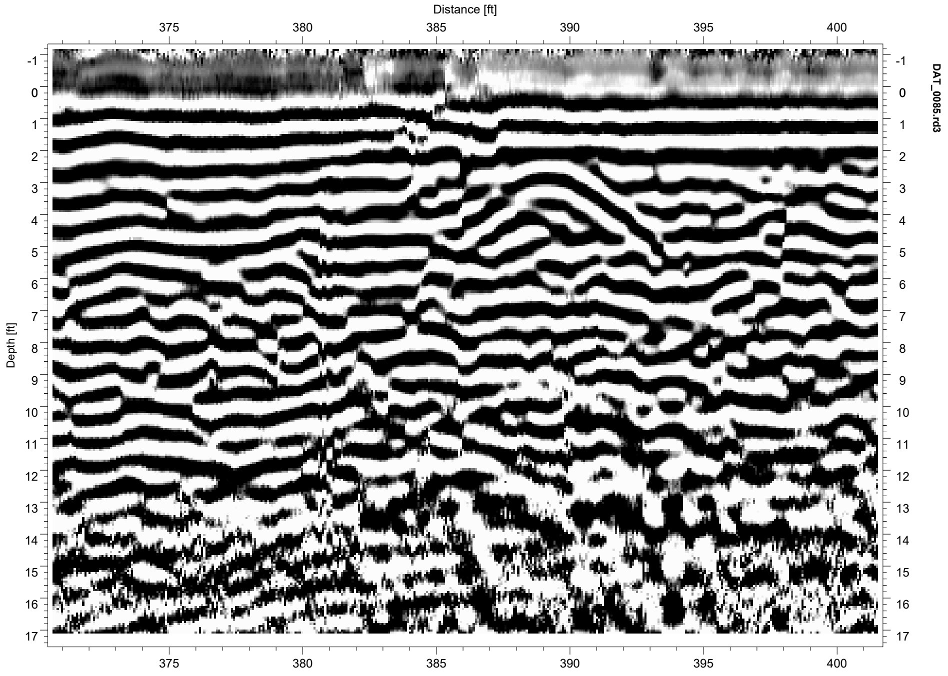

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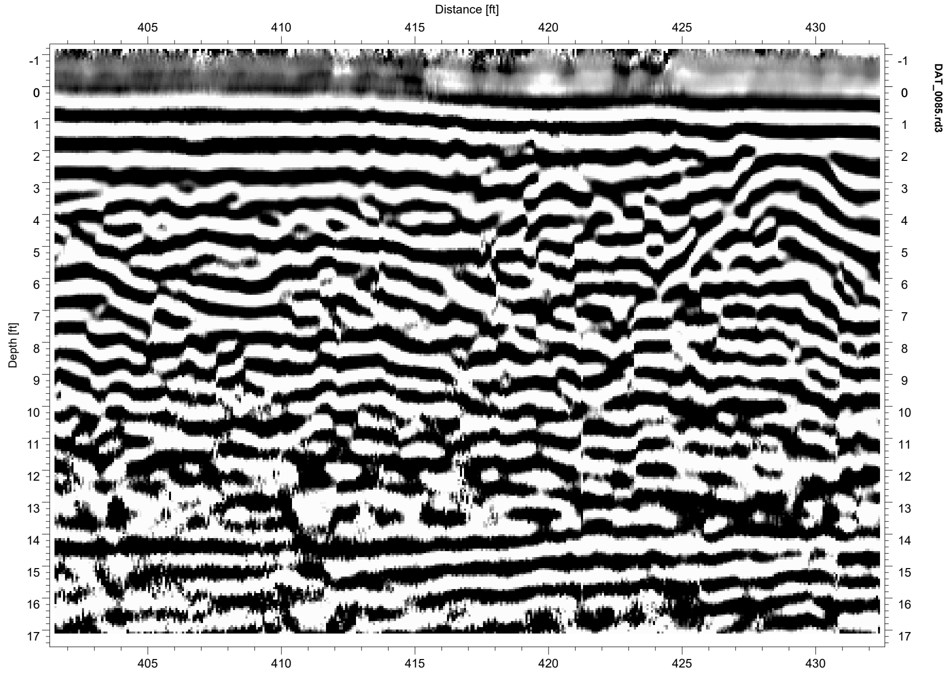

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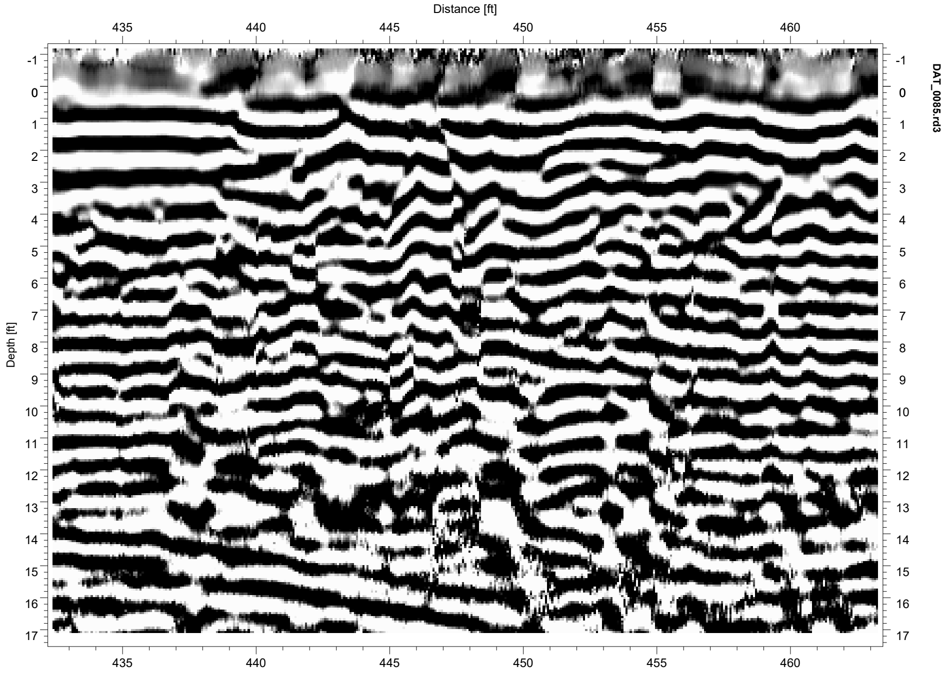

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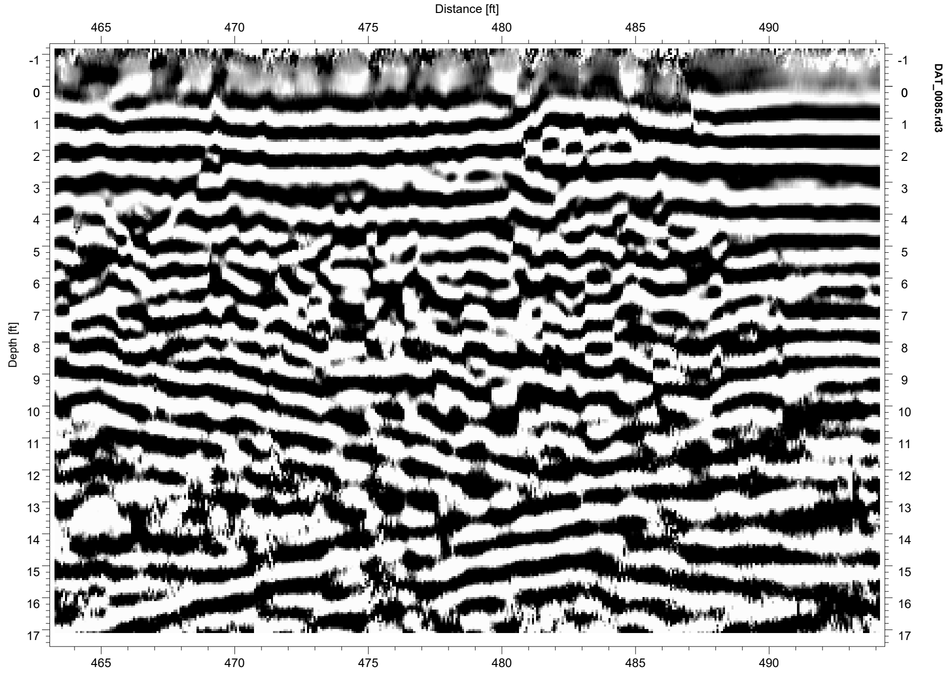

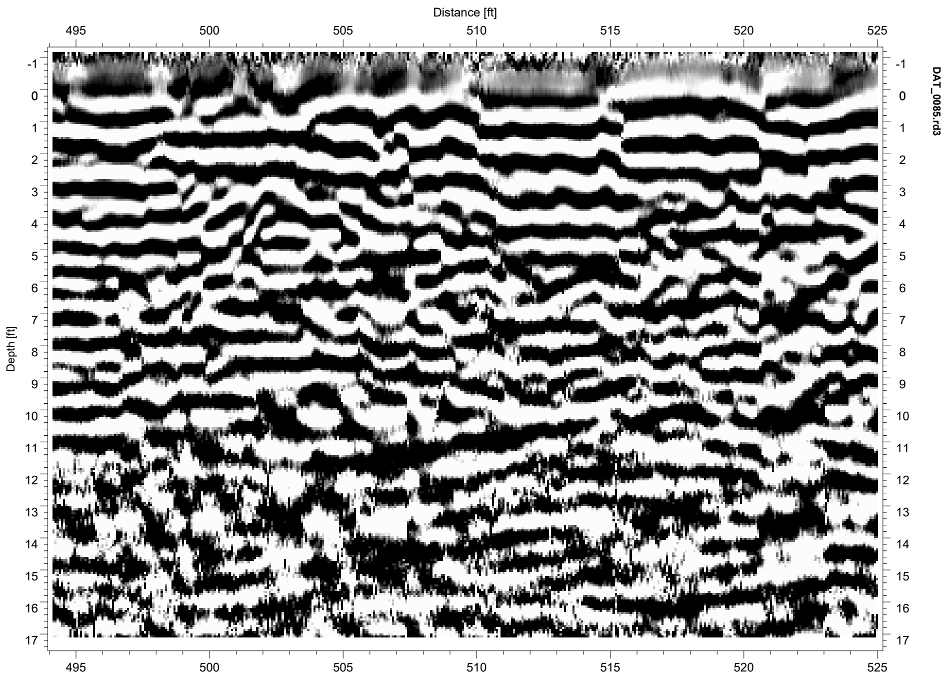
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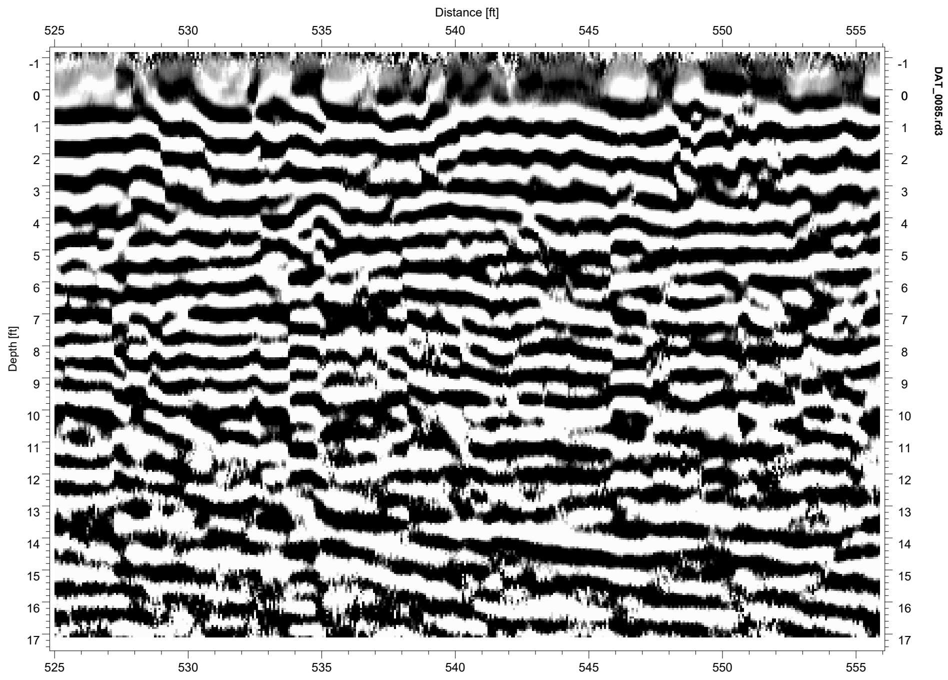
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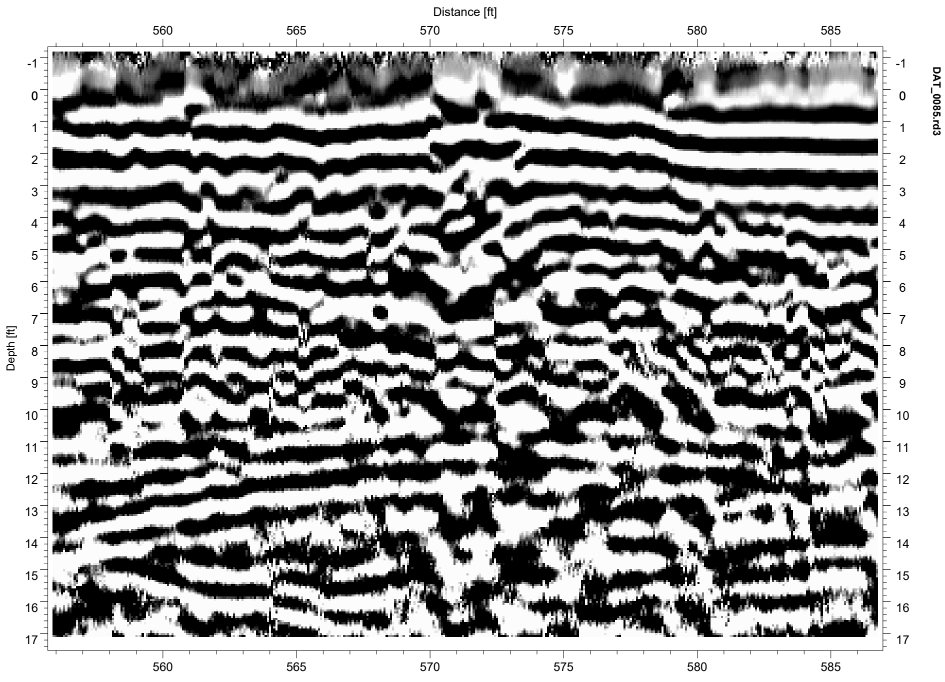
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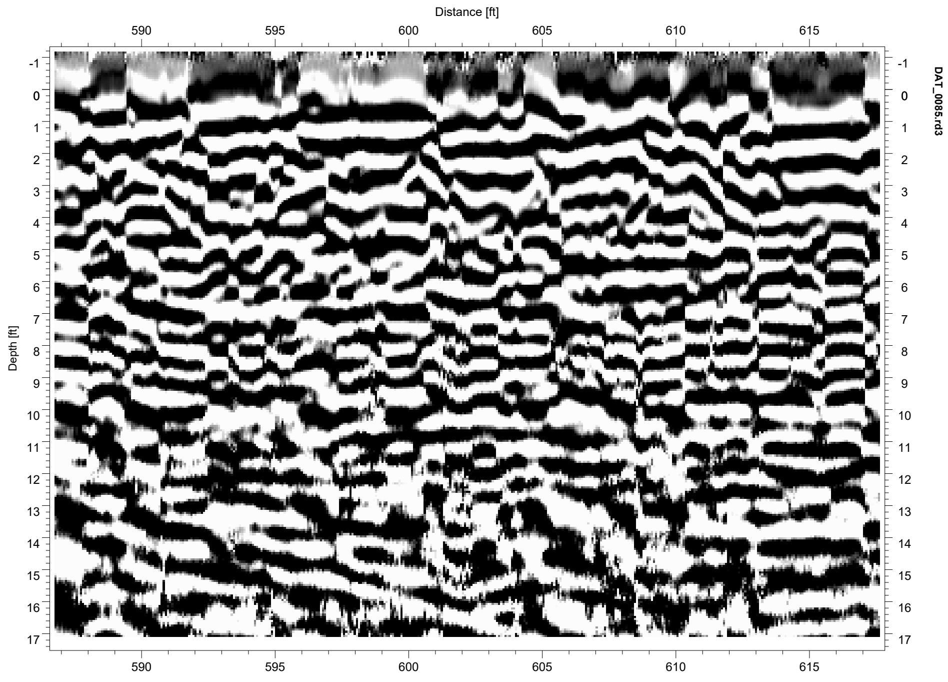
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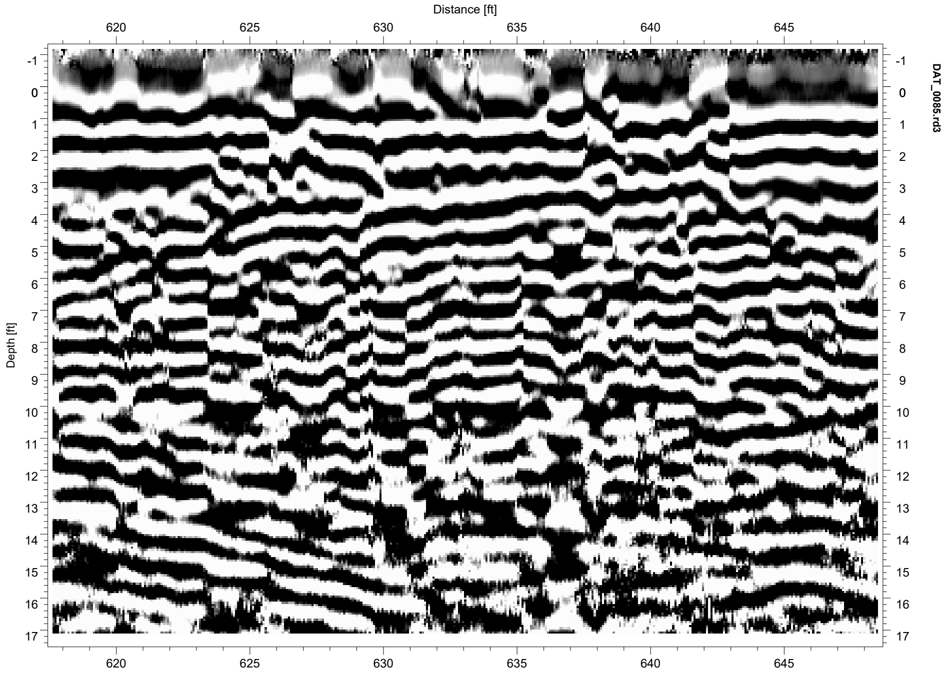

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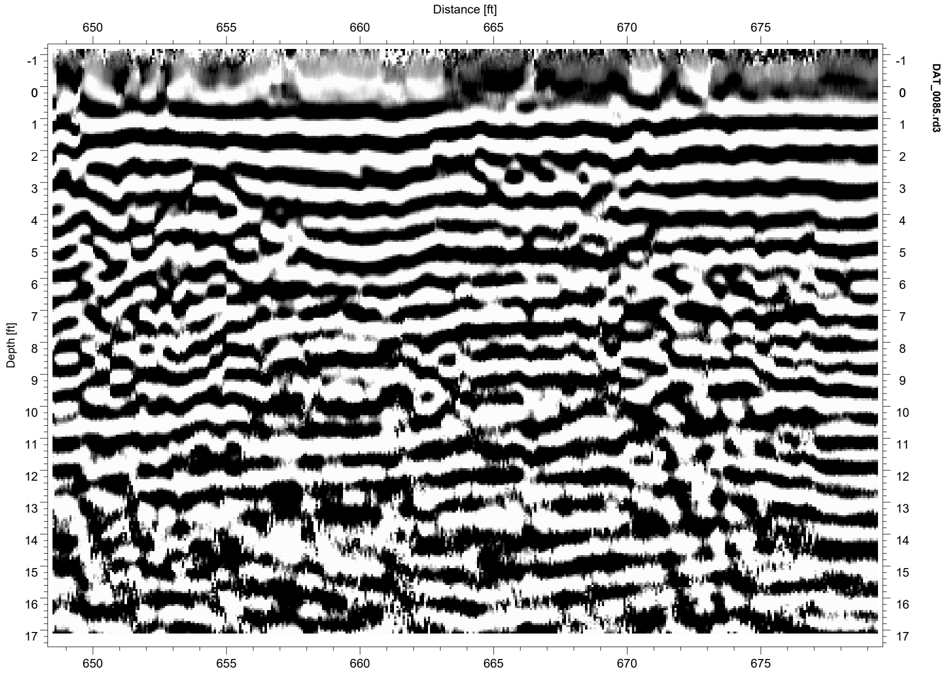

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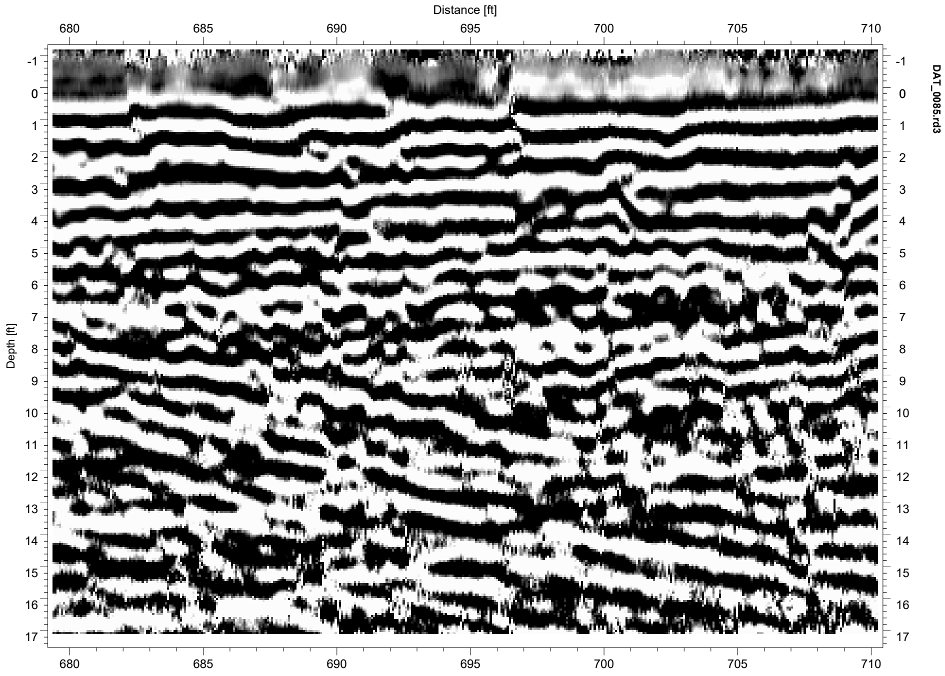

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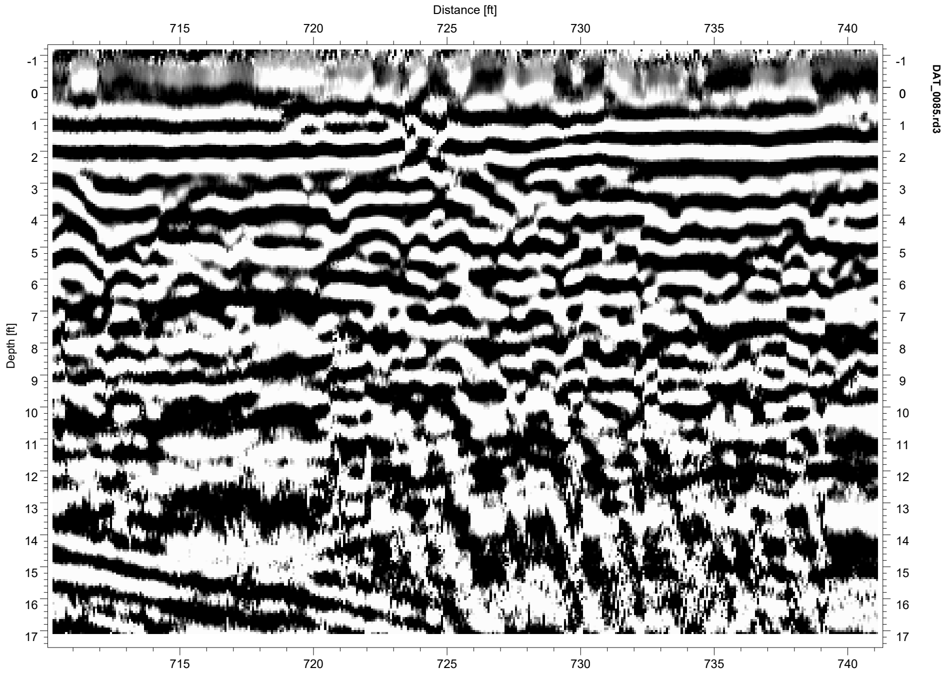

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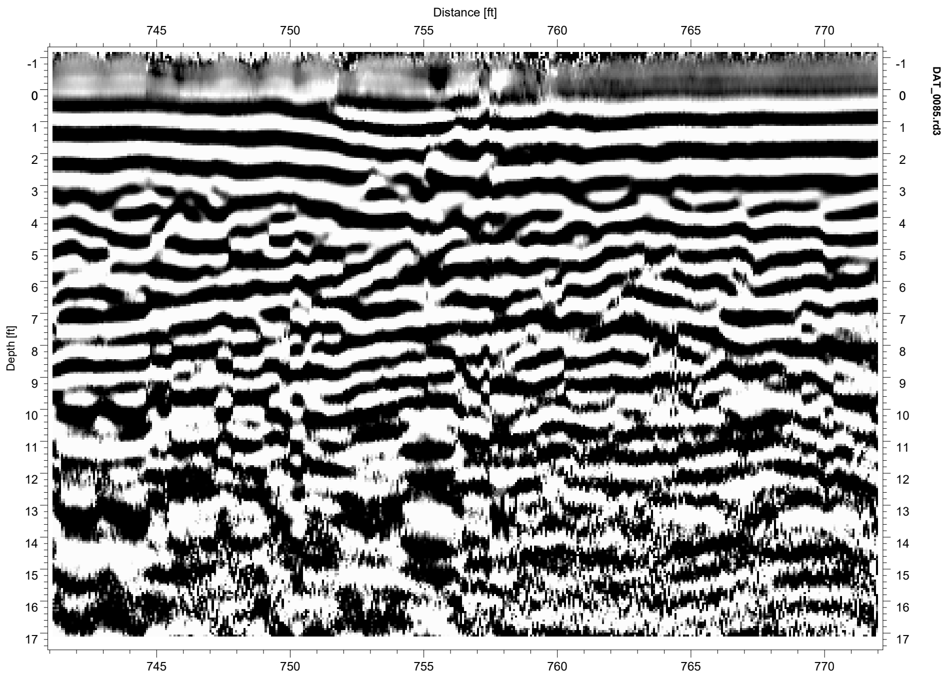

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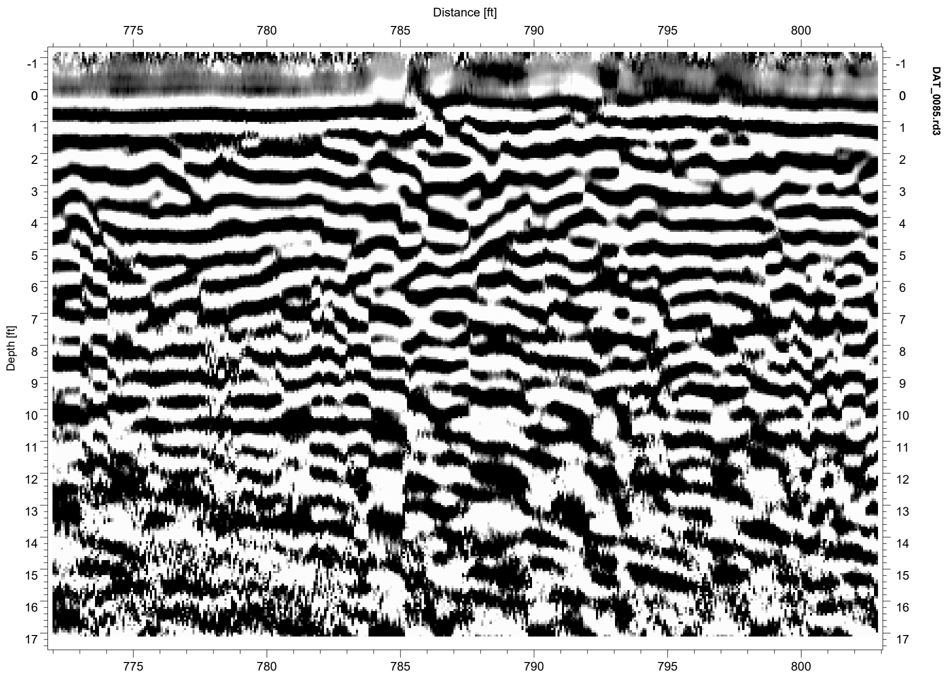

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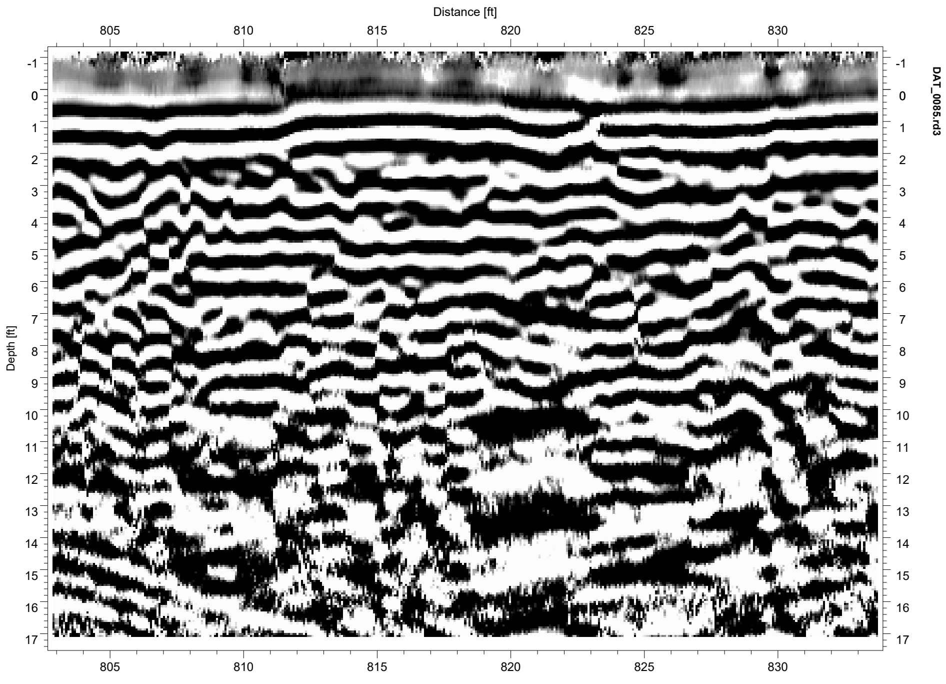

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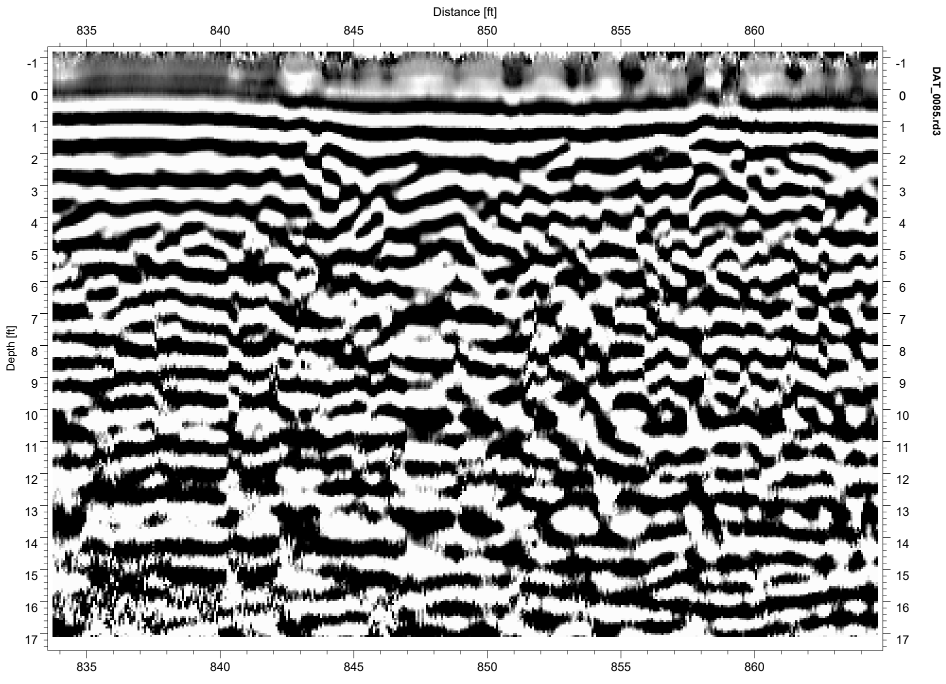

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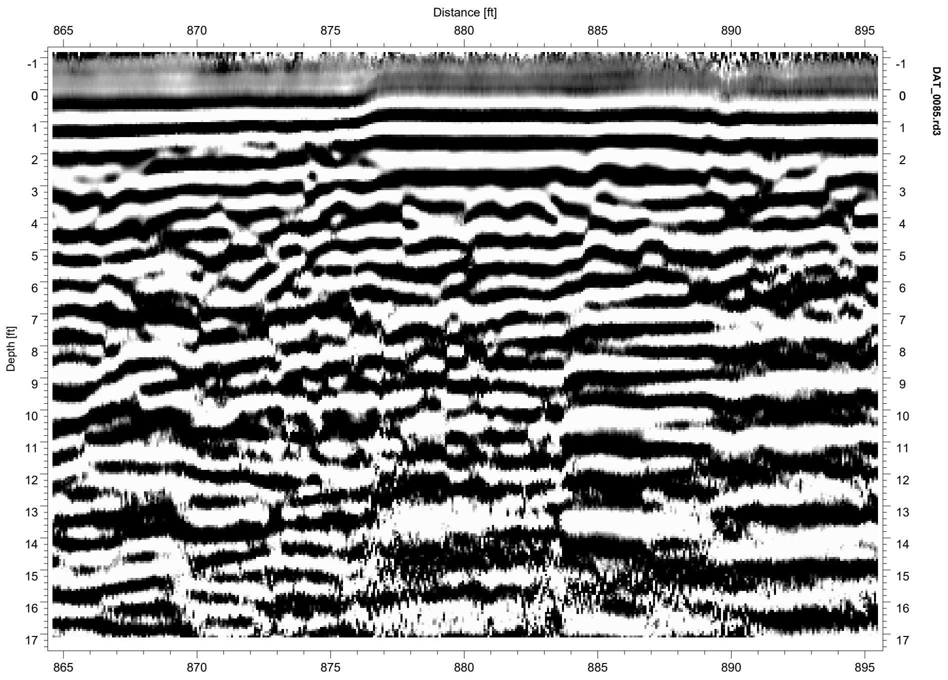

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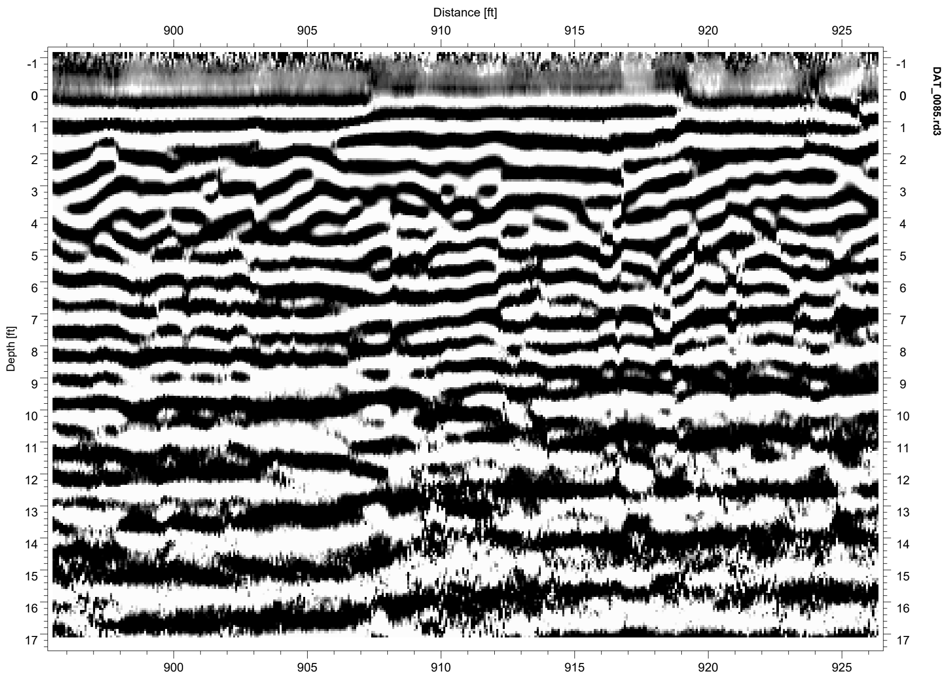

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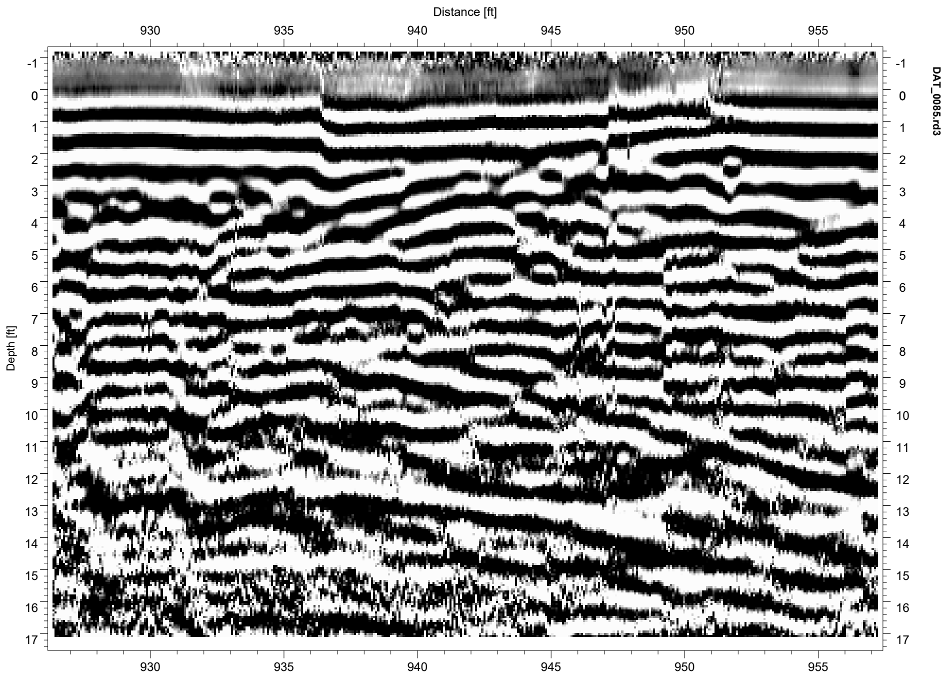

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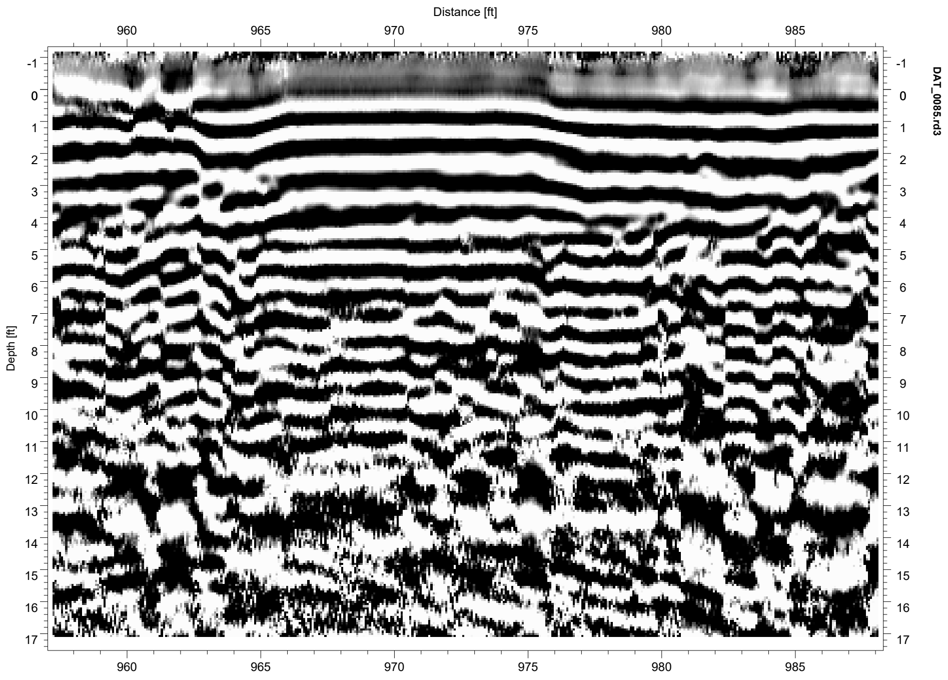

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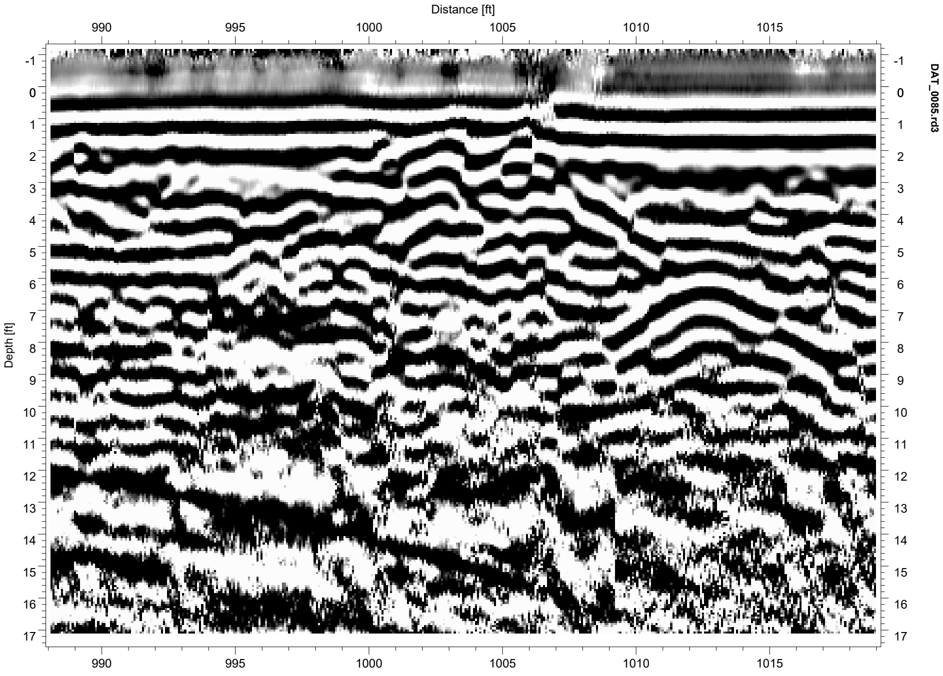

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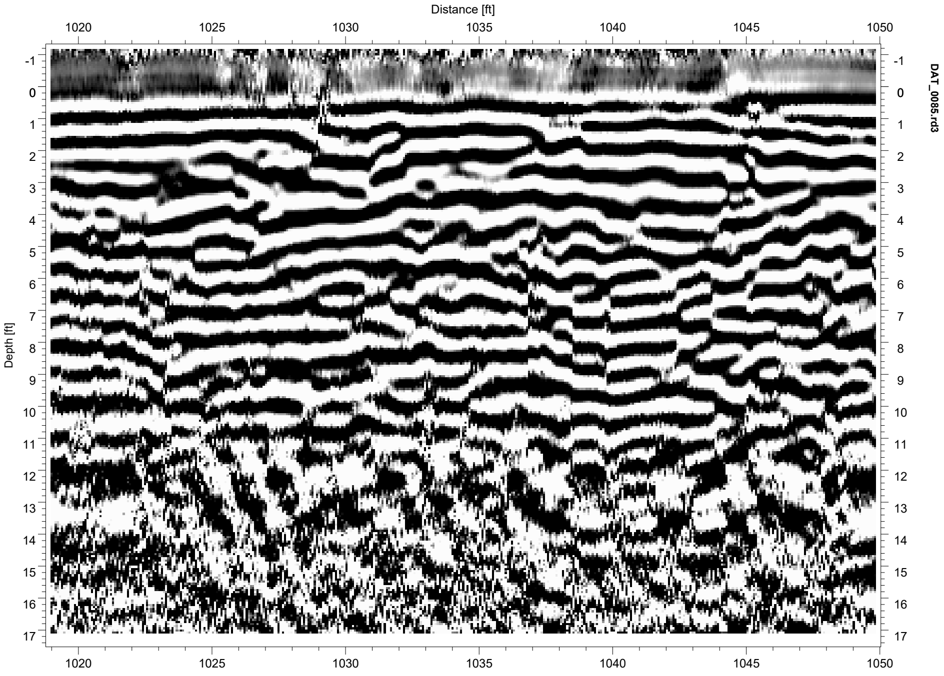

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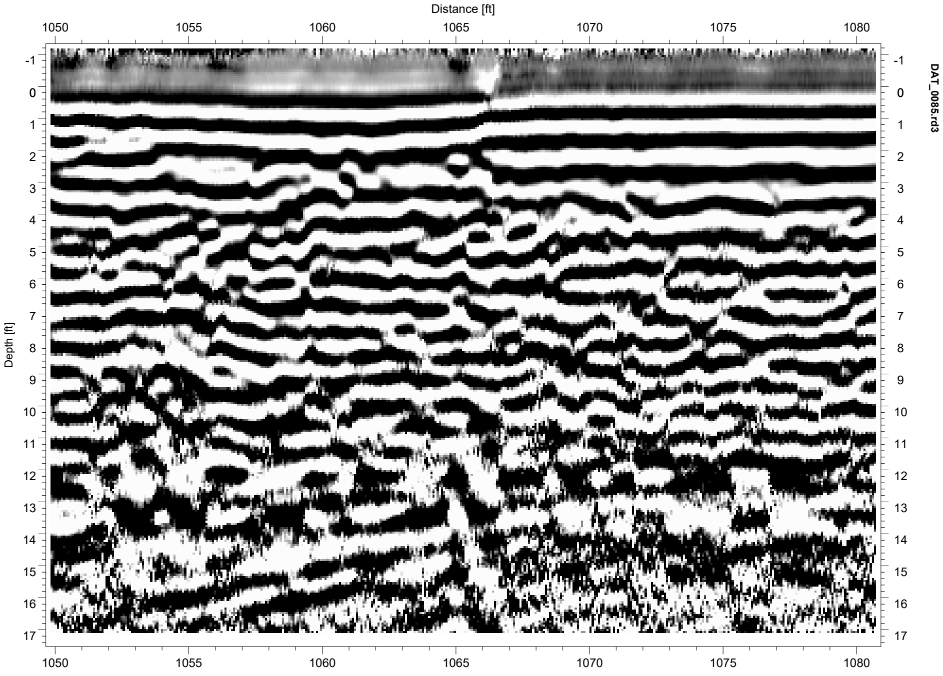

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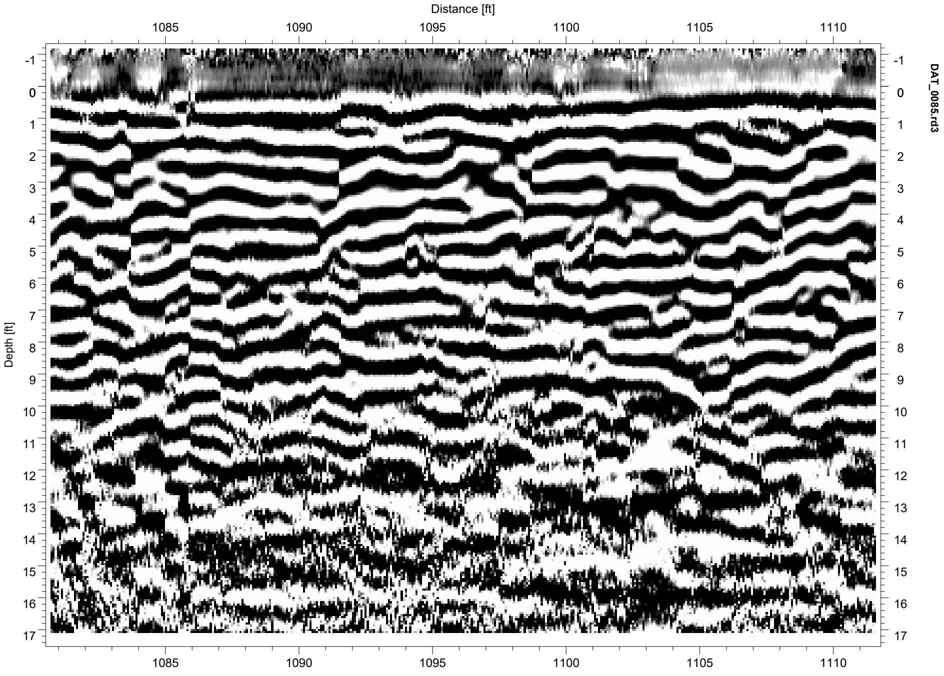

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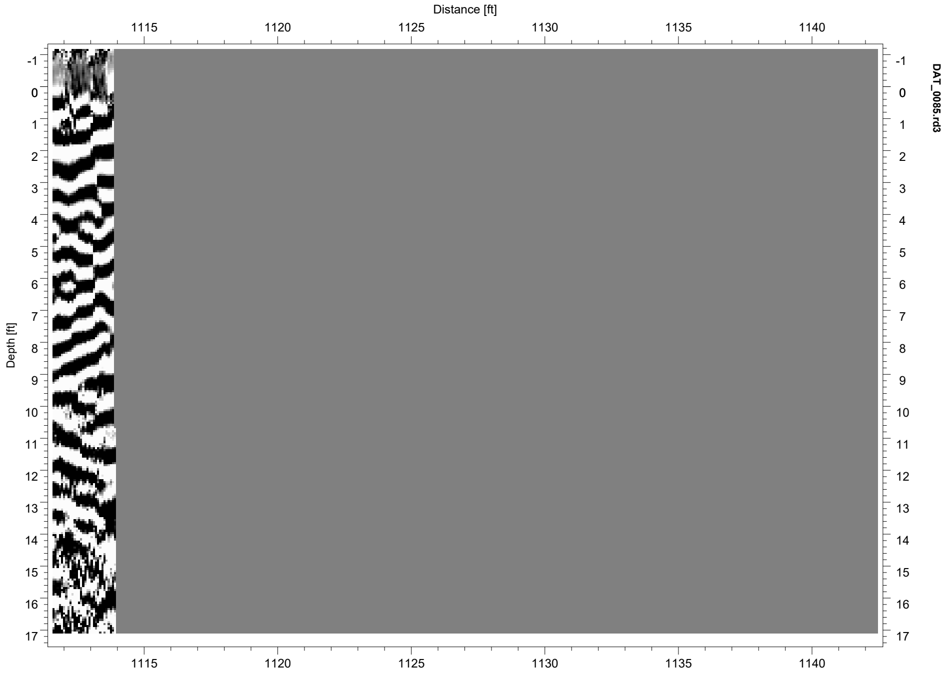

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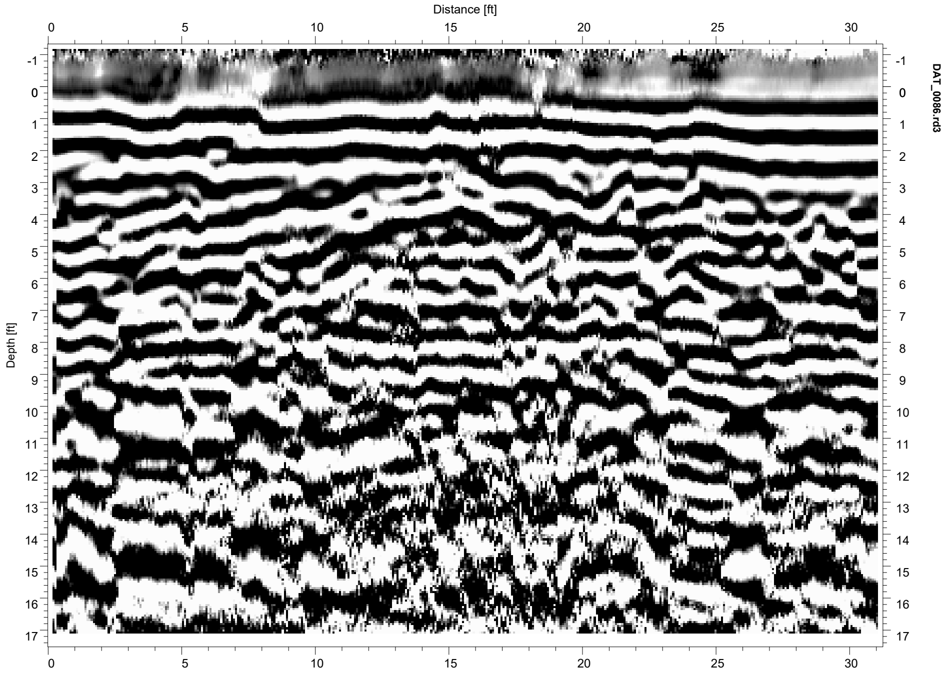

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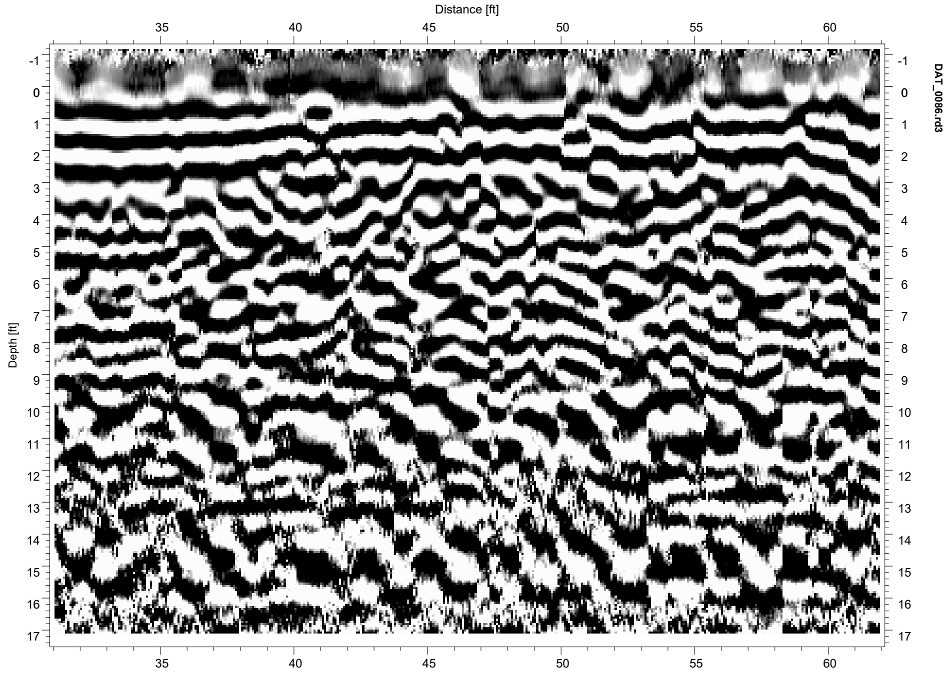

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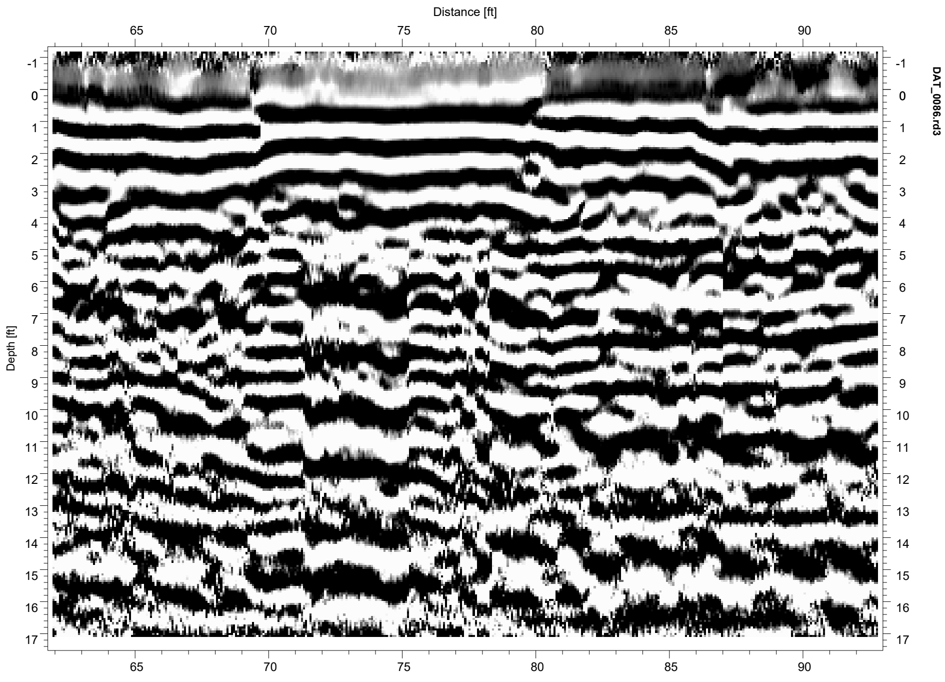

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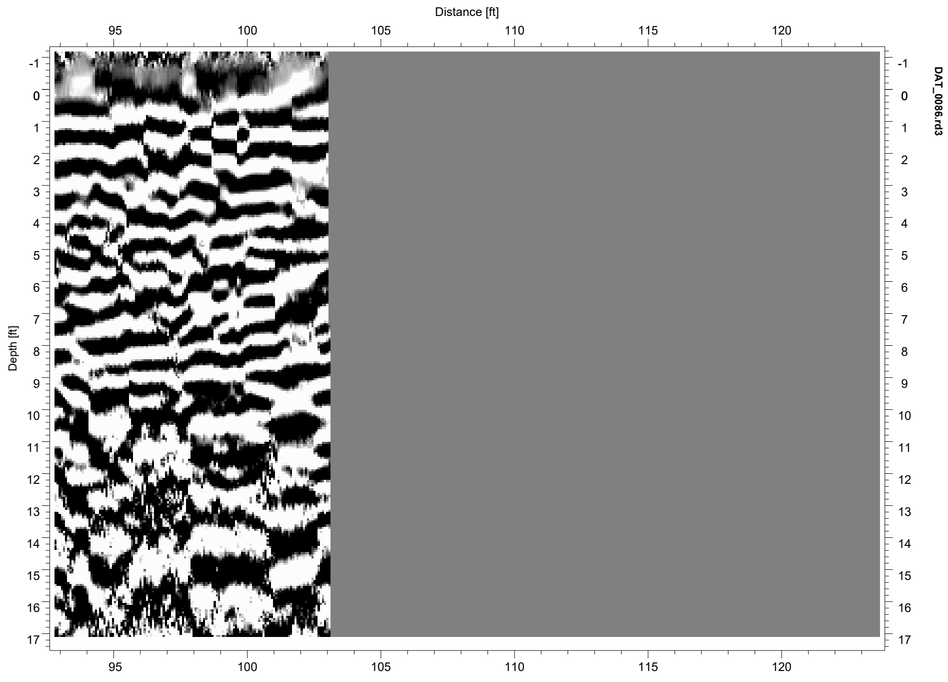

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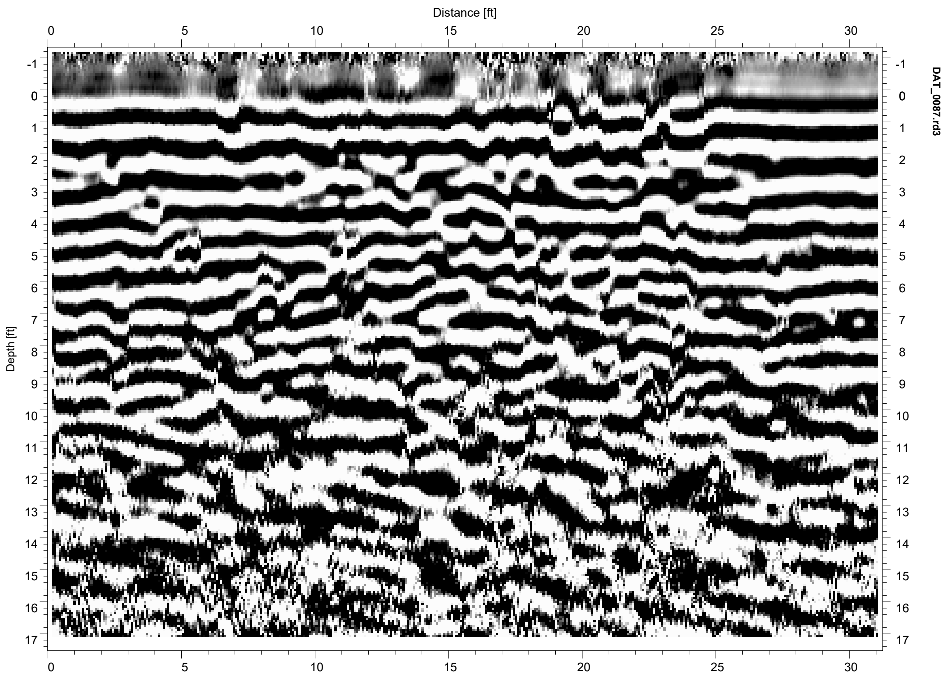

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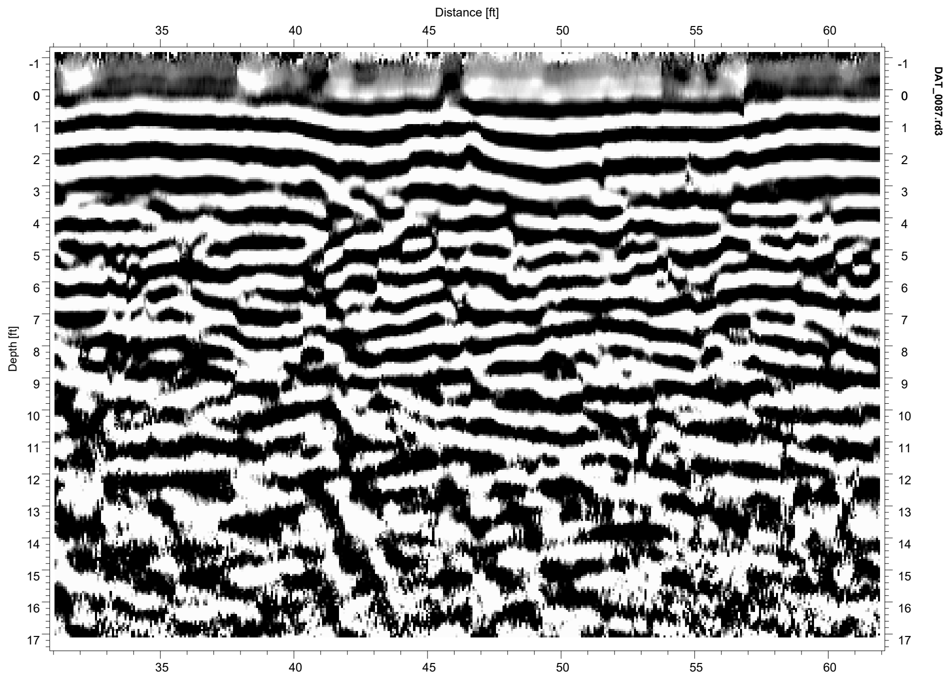

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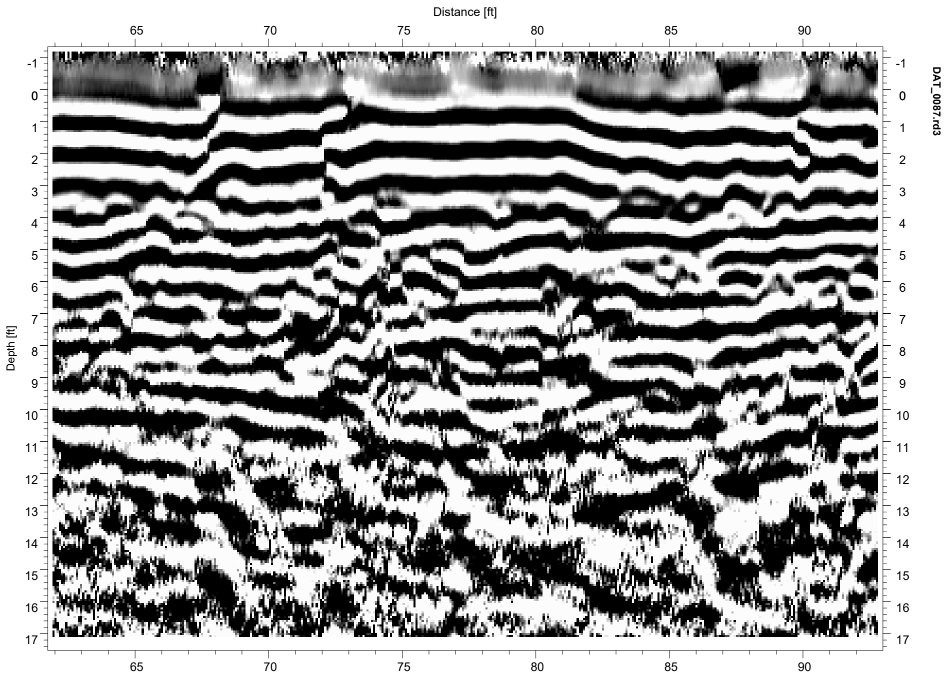

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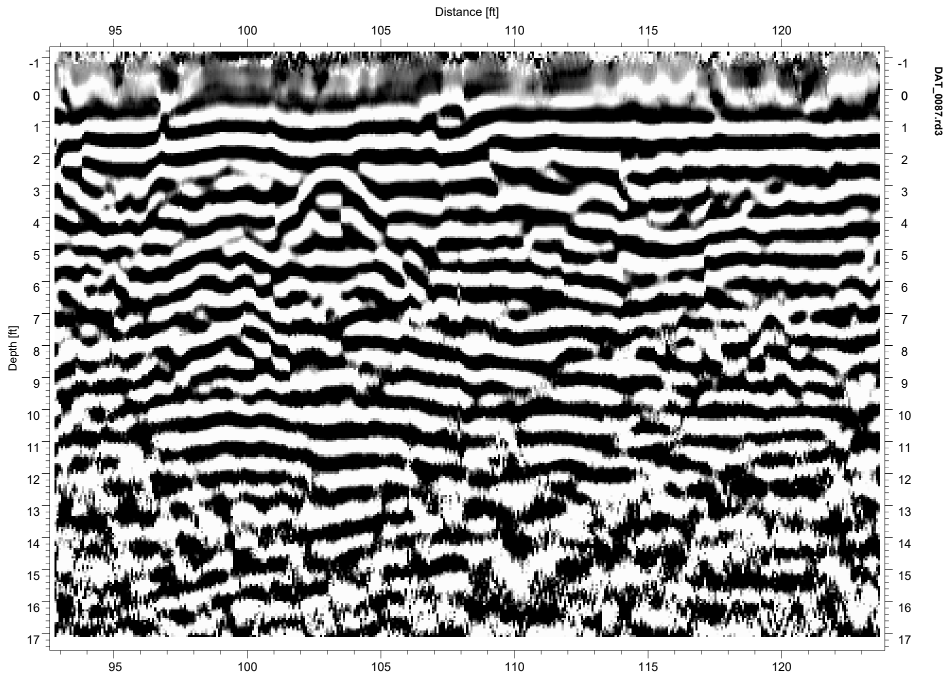

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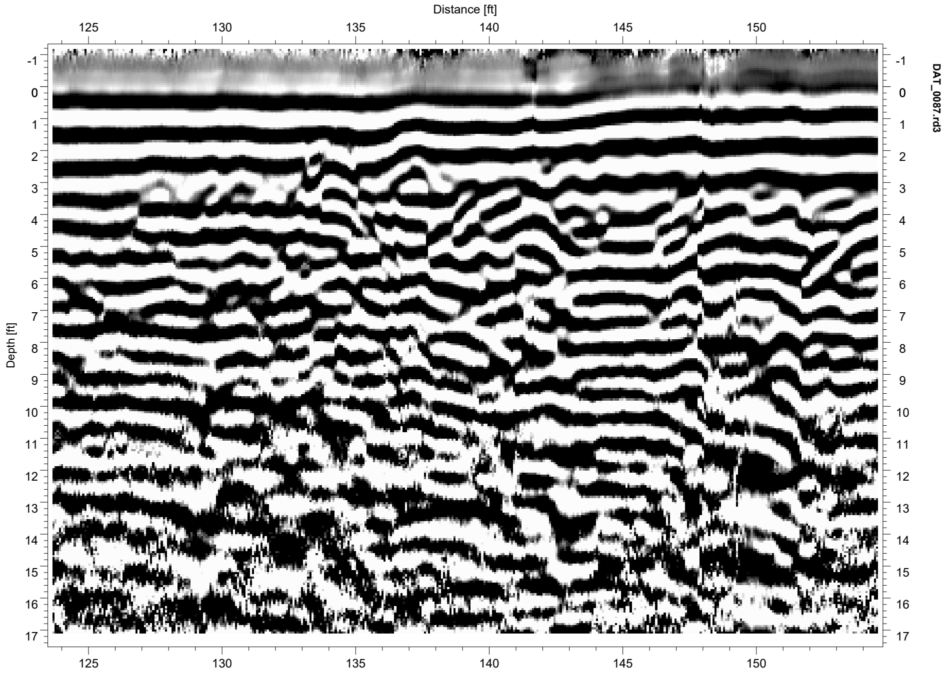

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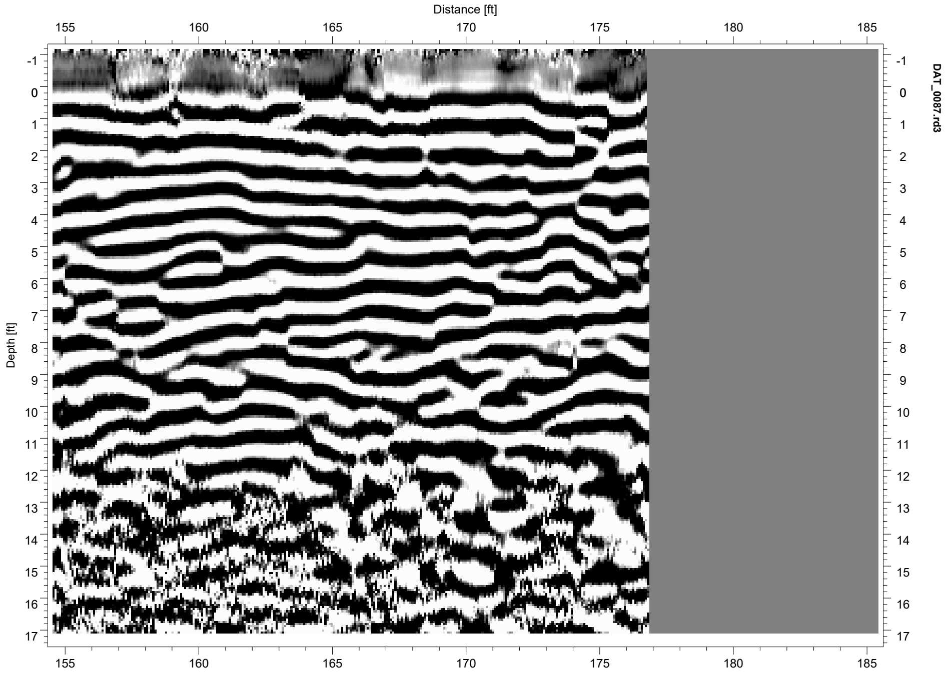

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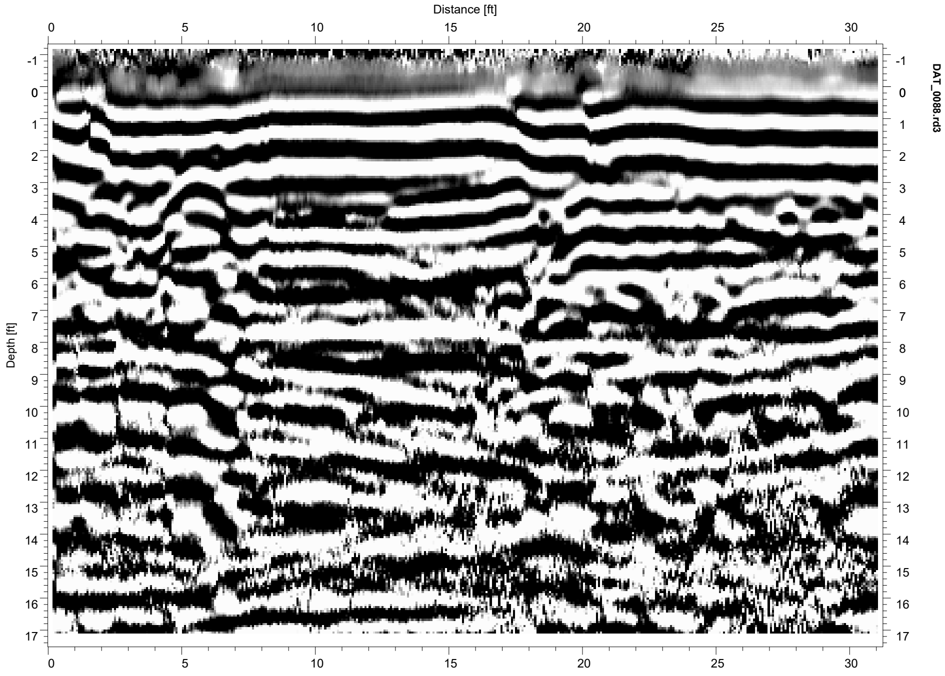

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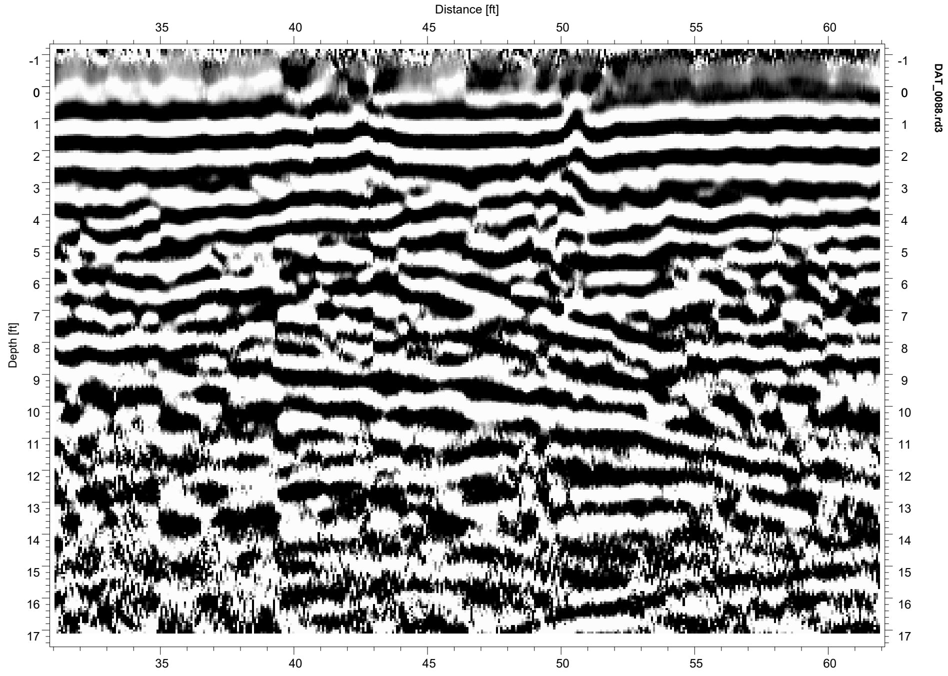

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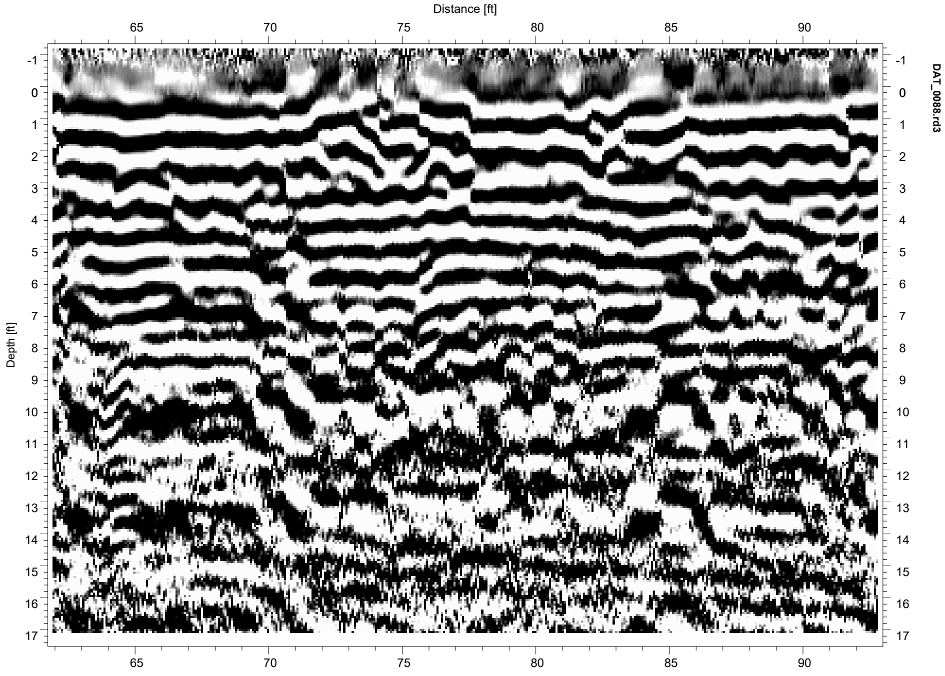

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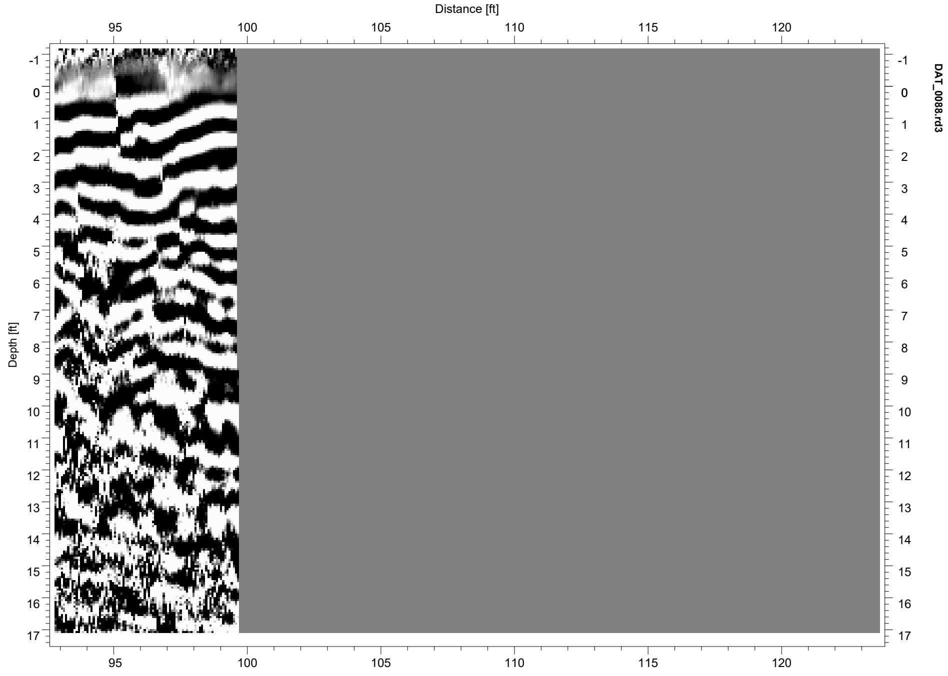

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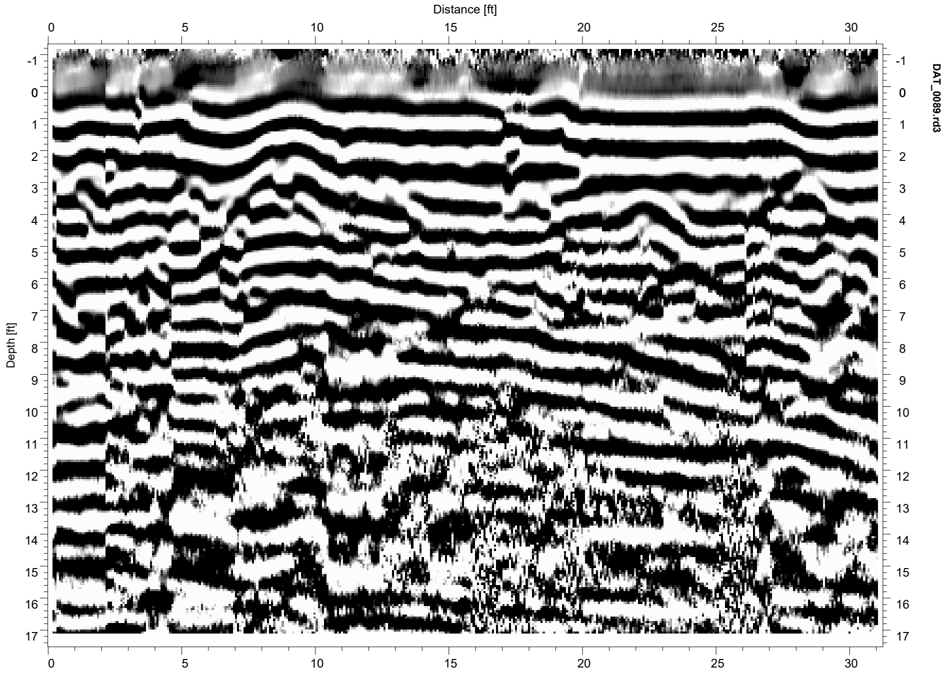

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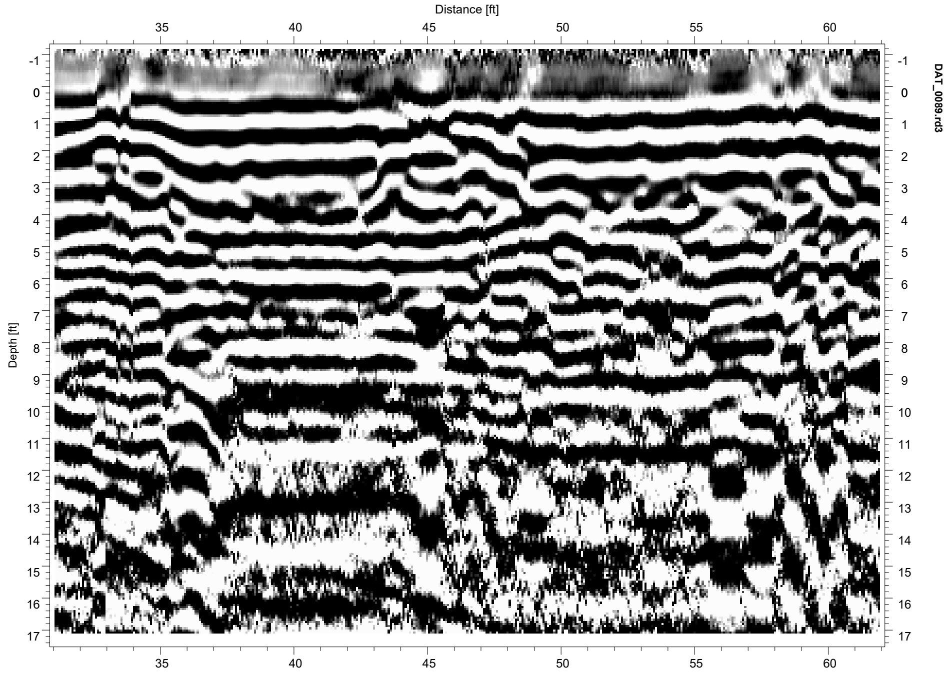

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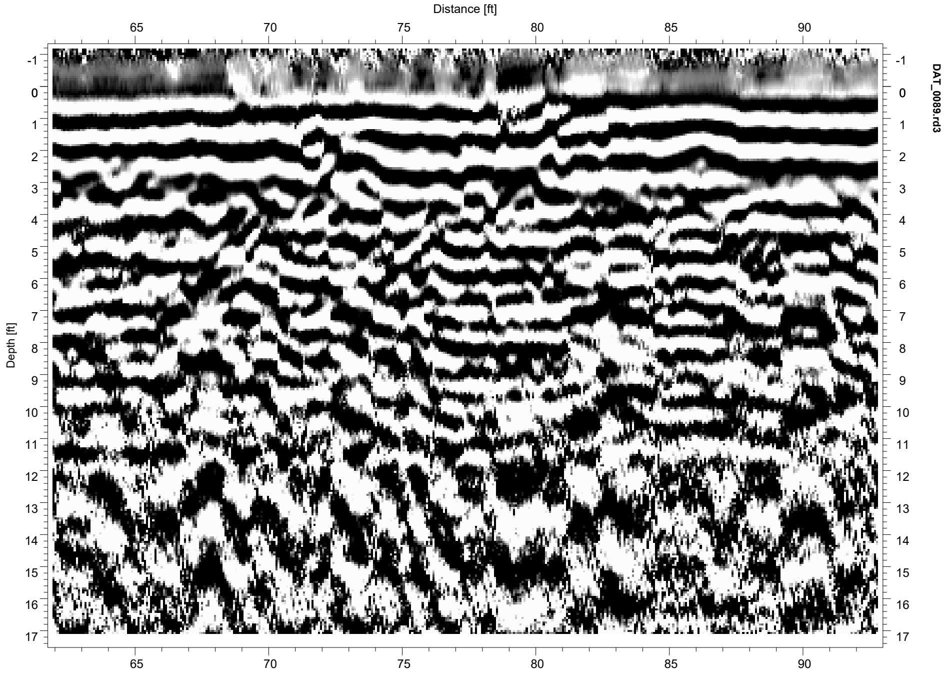

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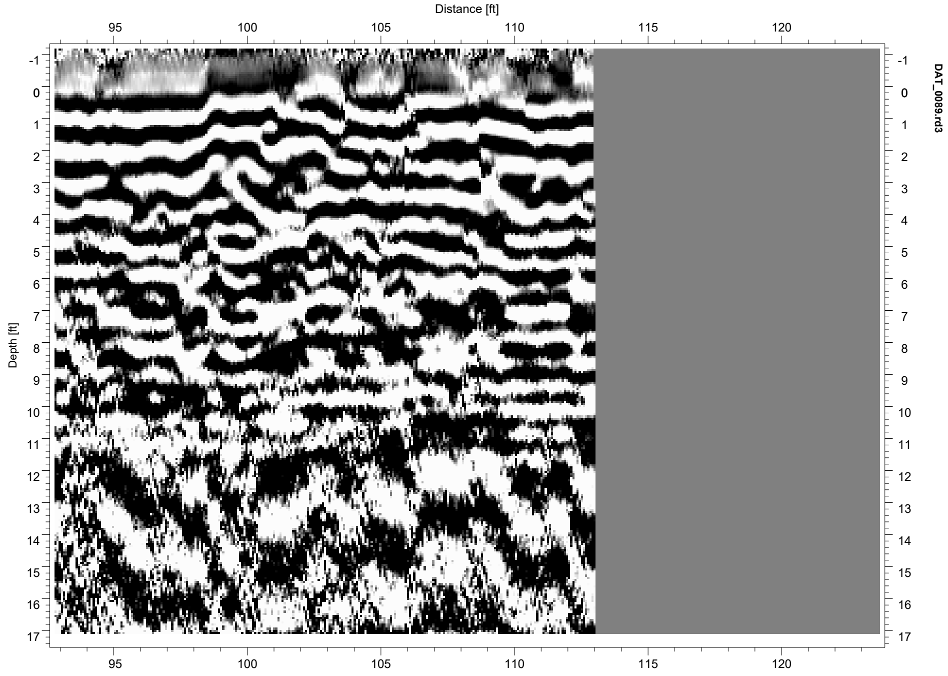

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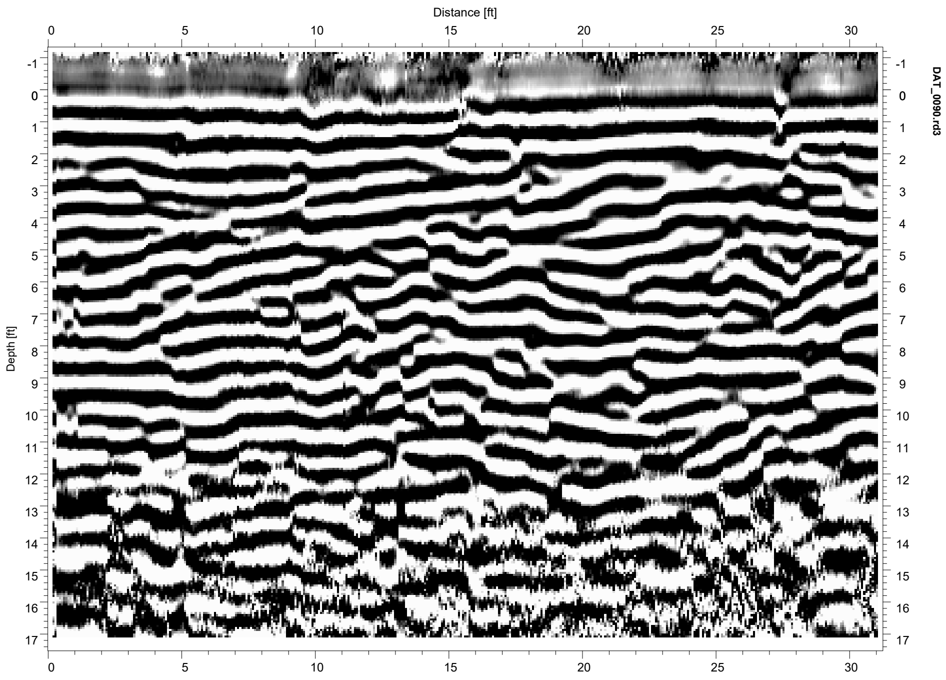

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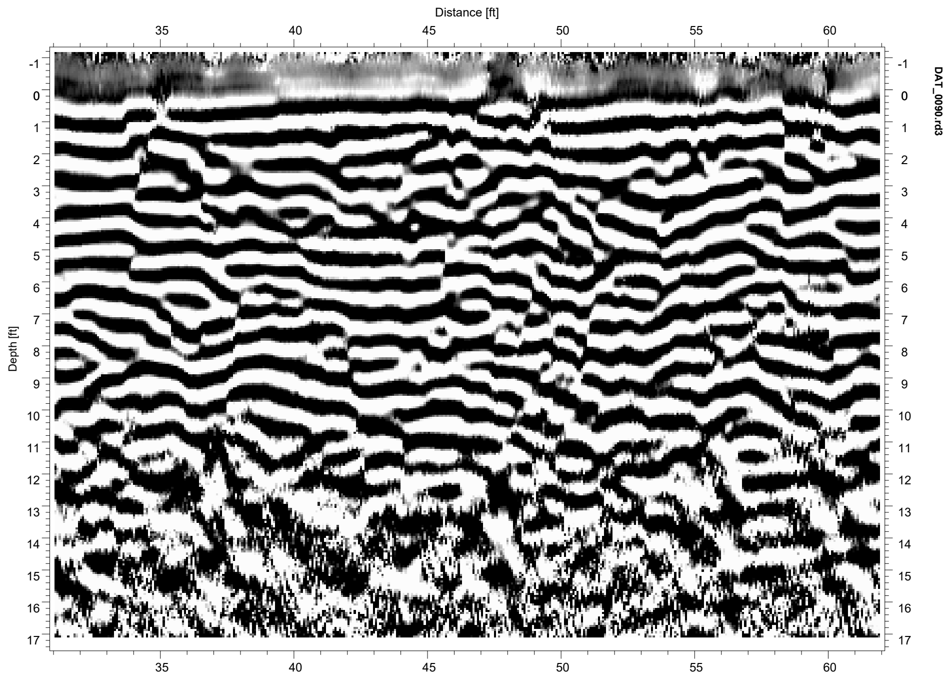

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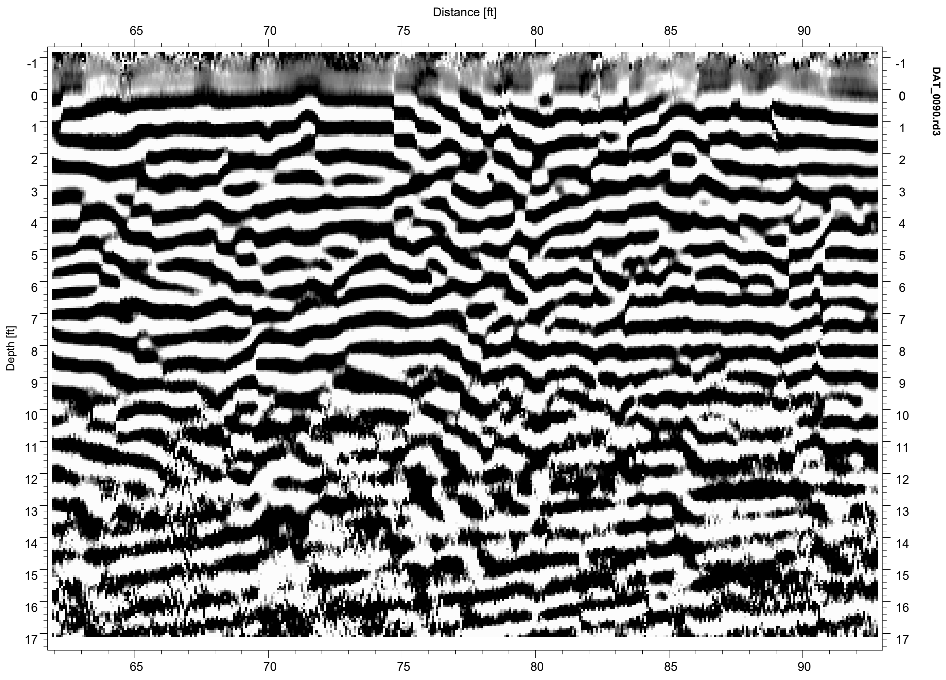

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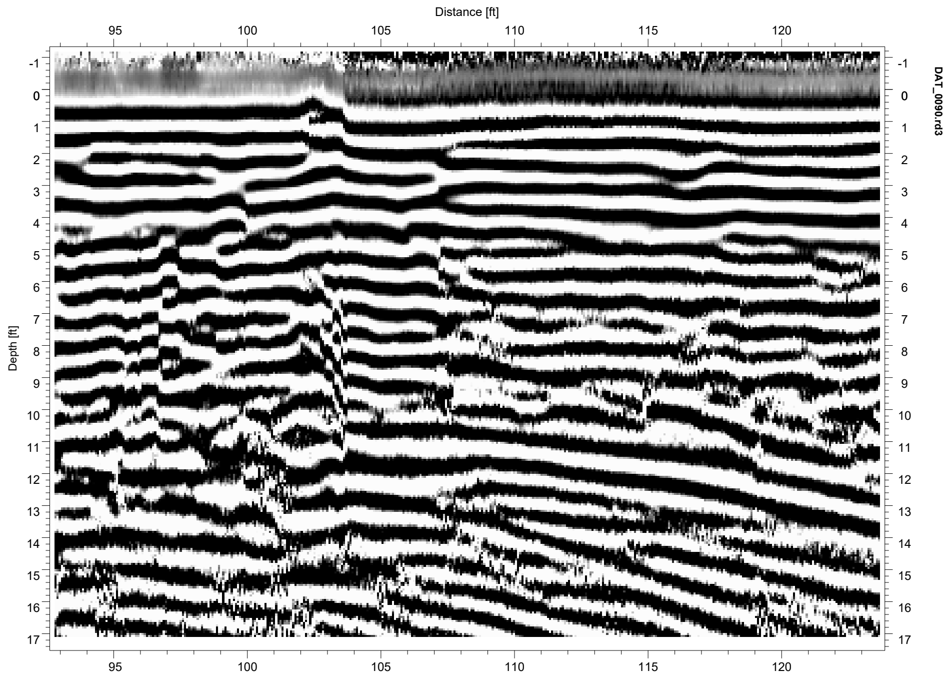

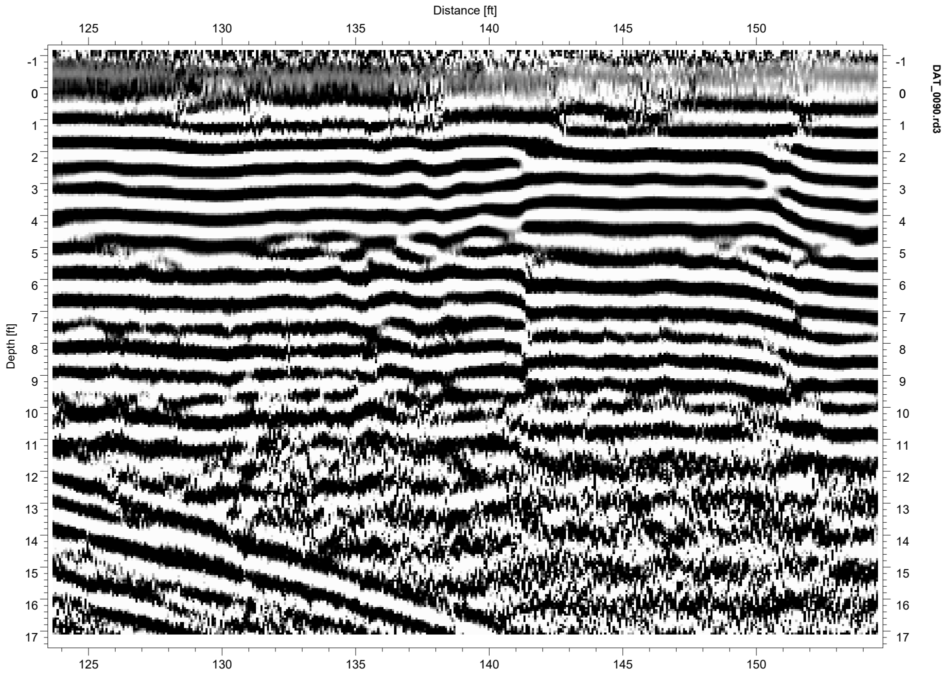
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Depth [ft]
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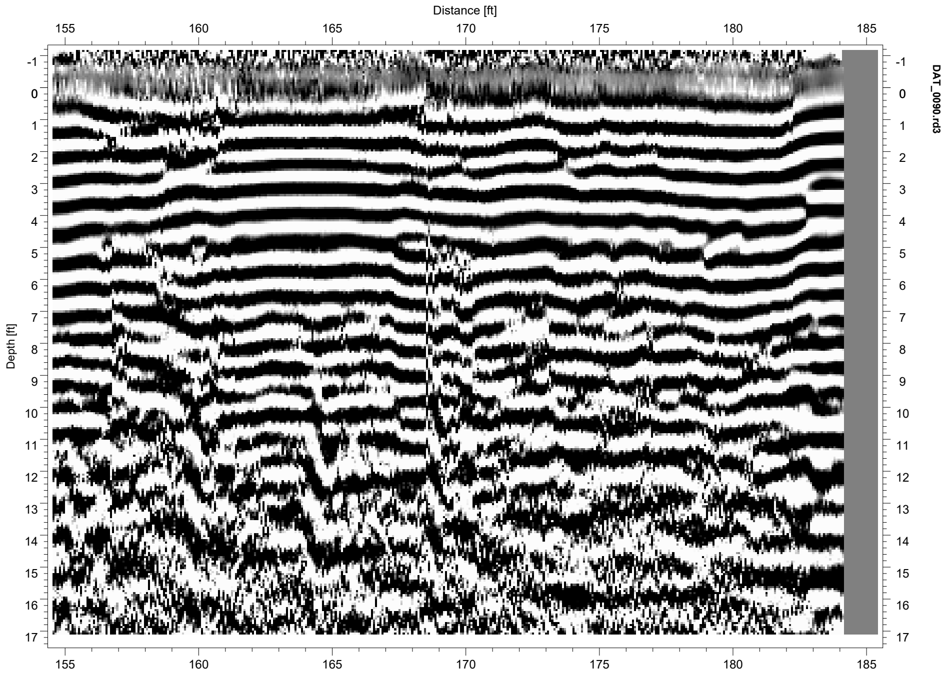
DAT_0090.rd3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Depth [ft]
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DAT_0090.rd3

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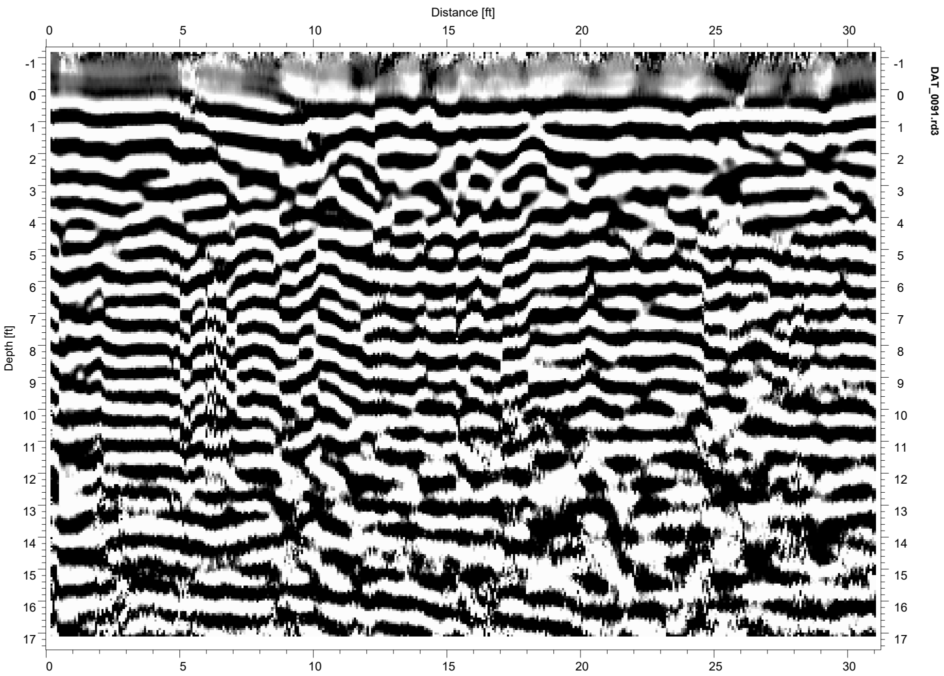

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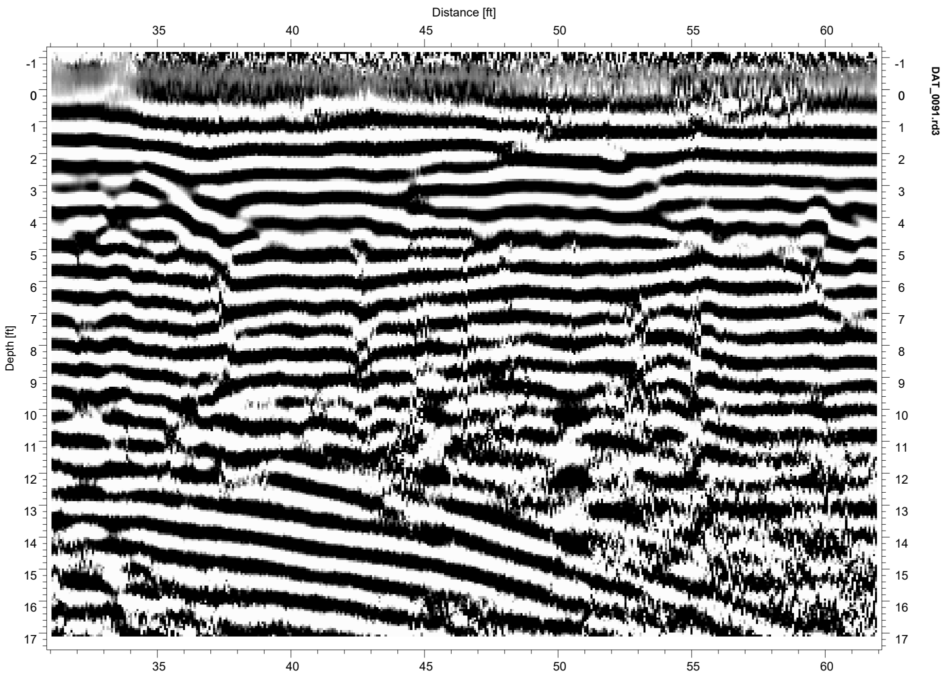

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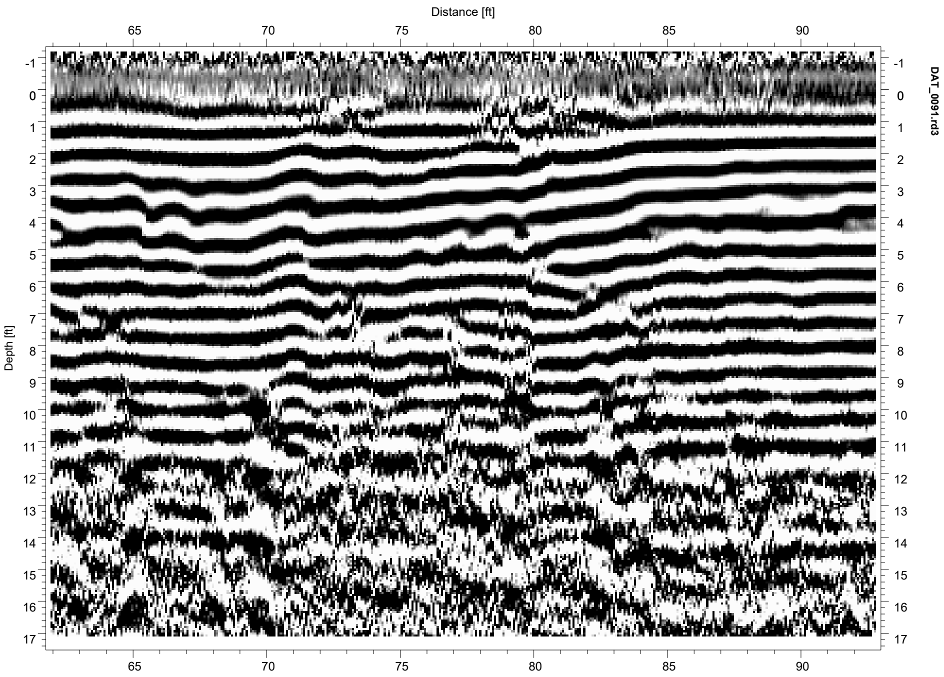

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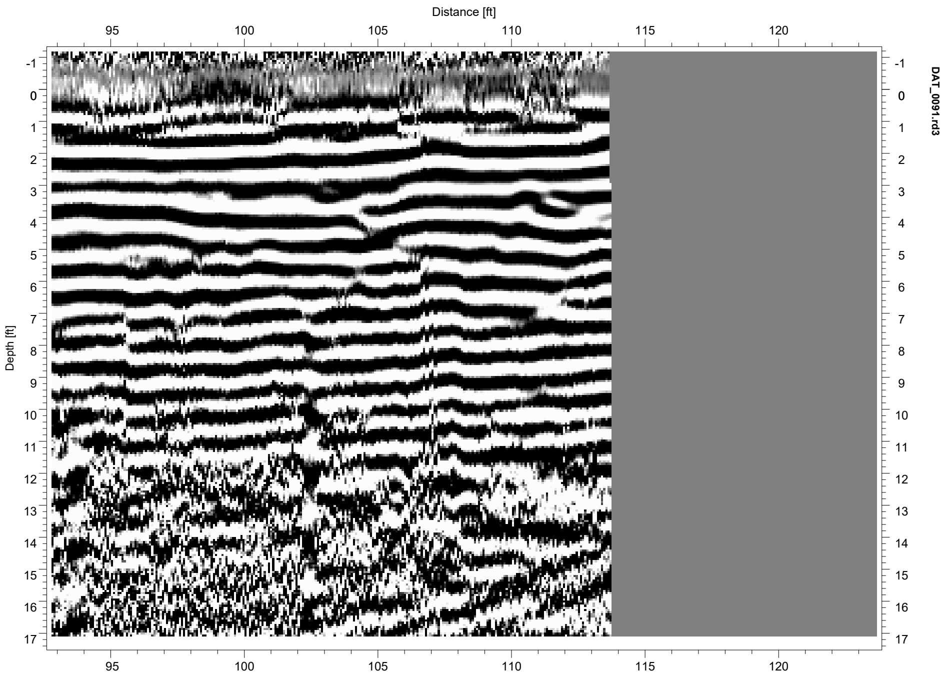

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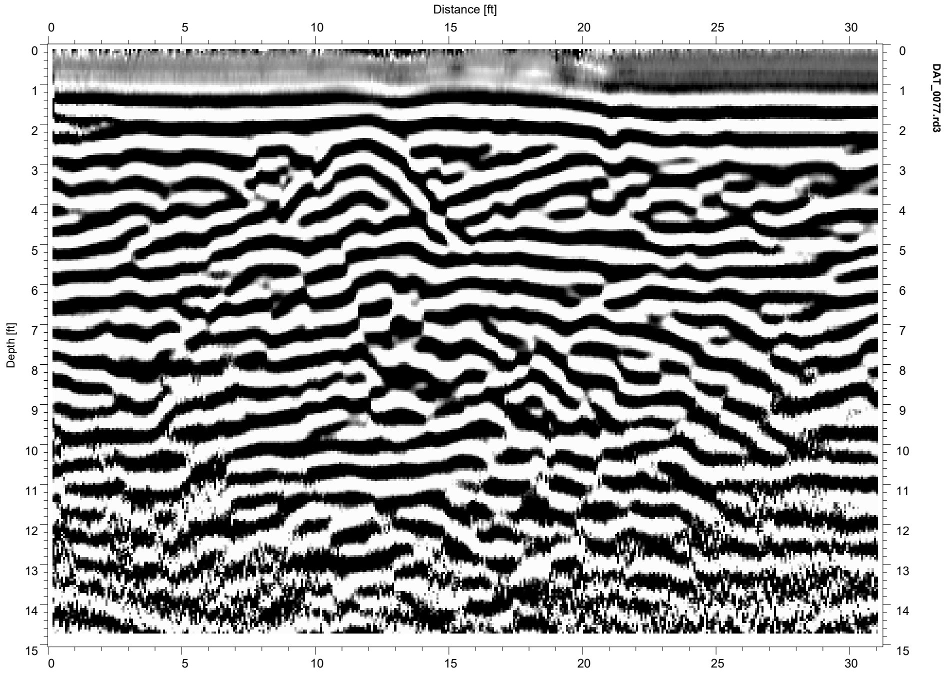

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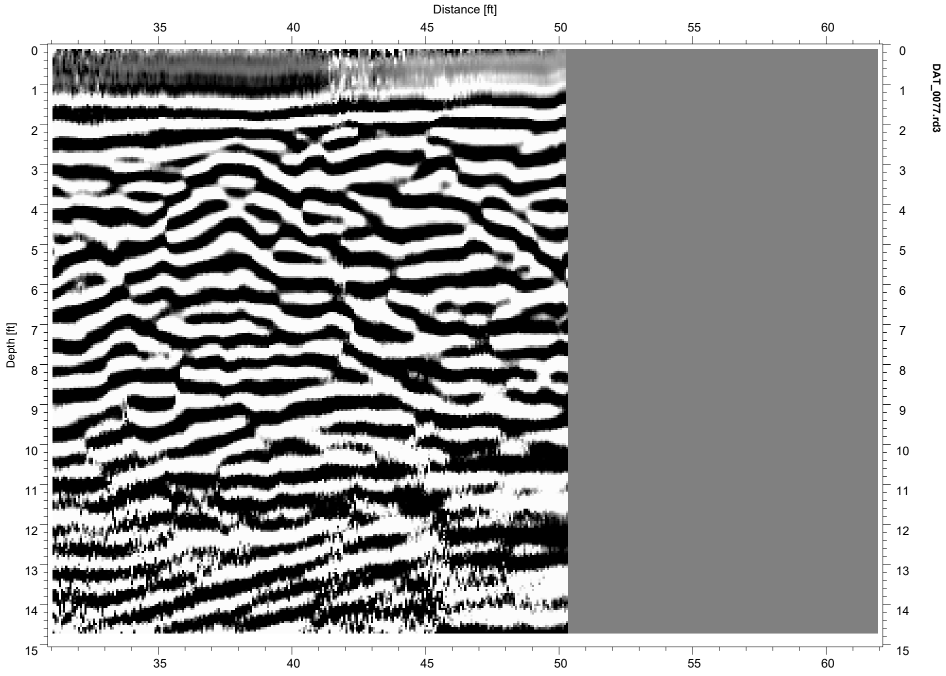

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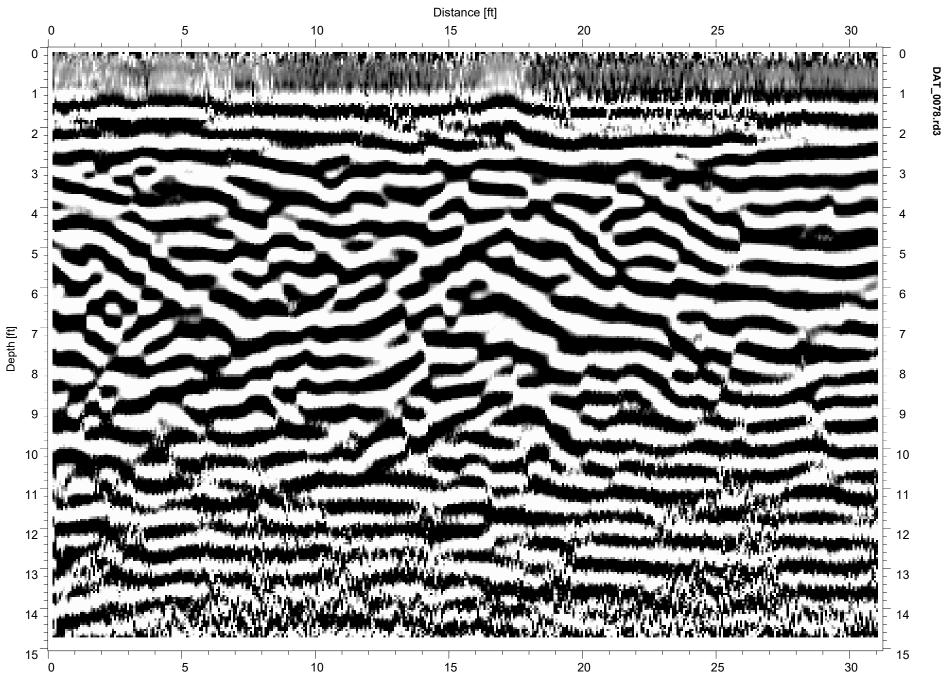

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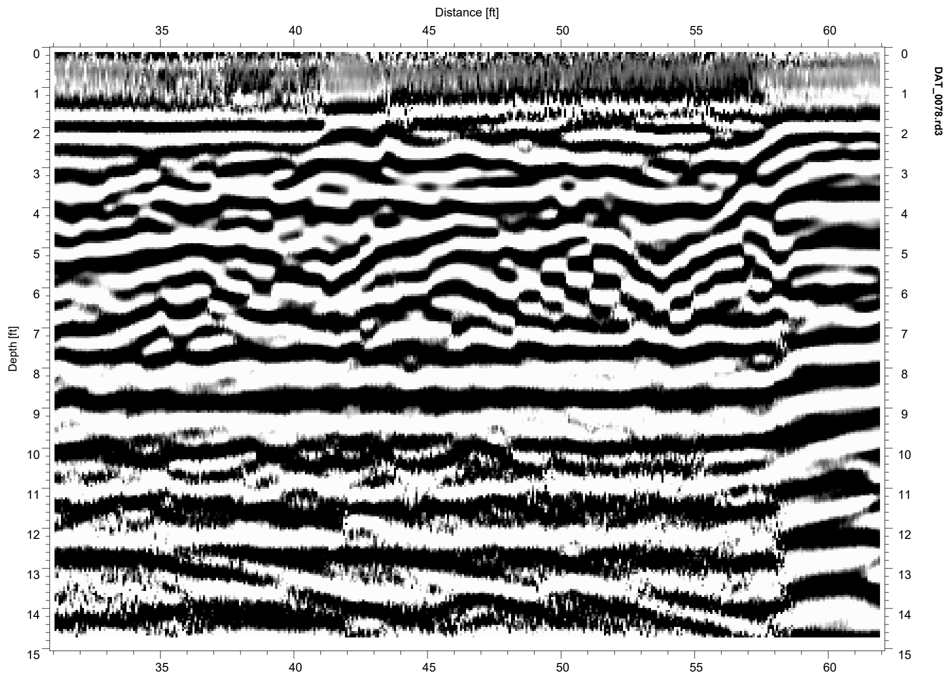

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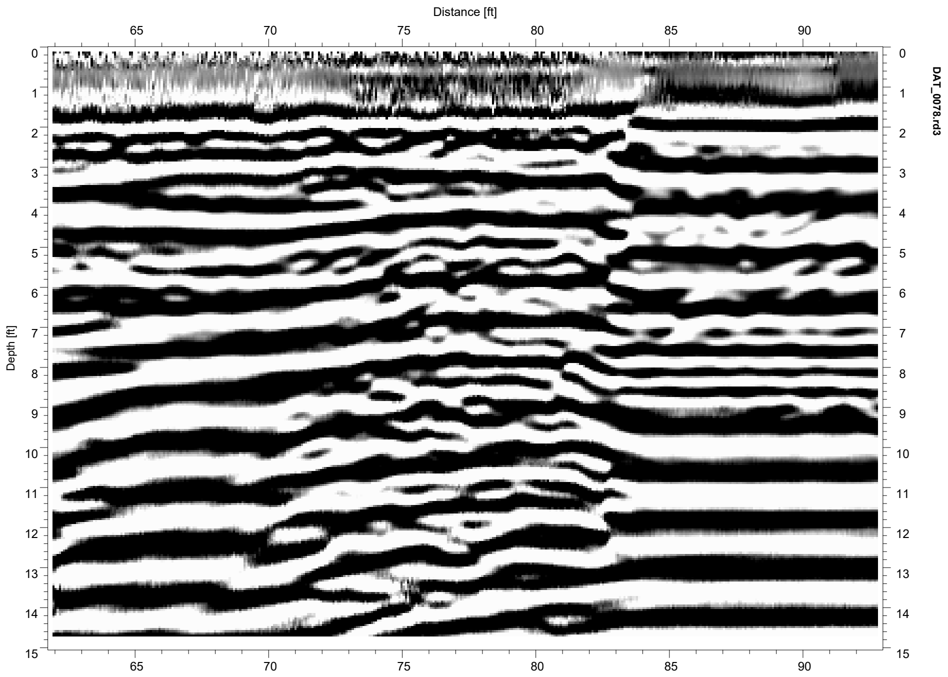

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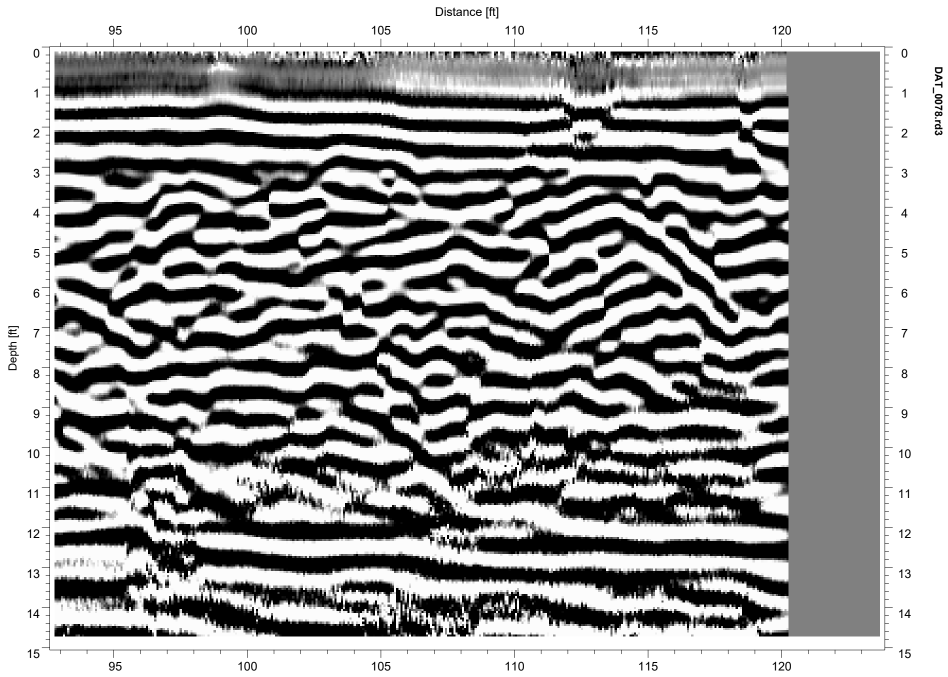

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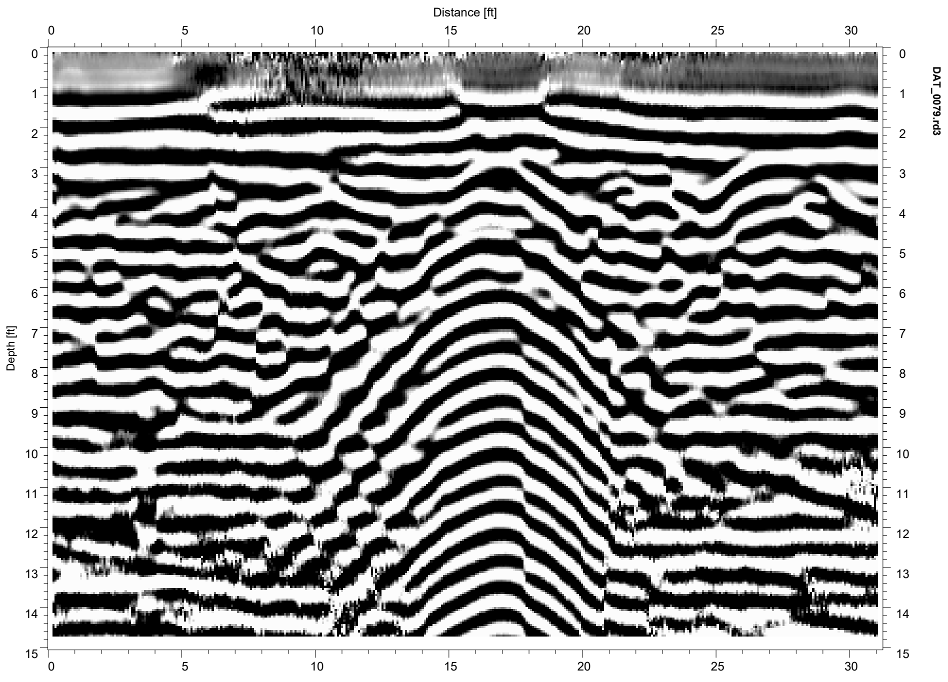

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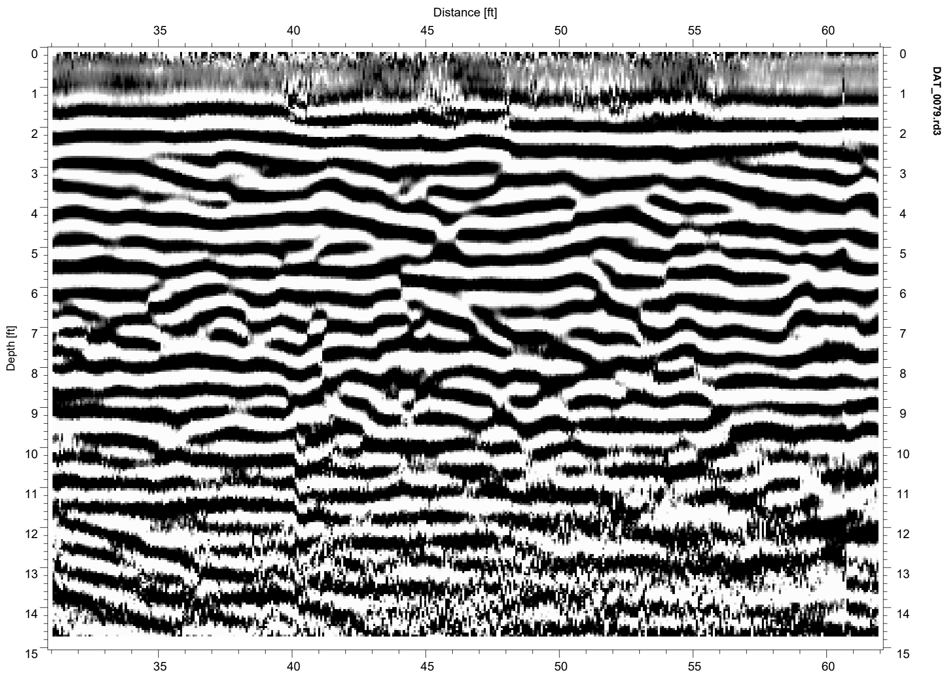

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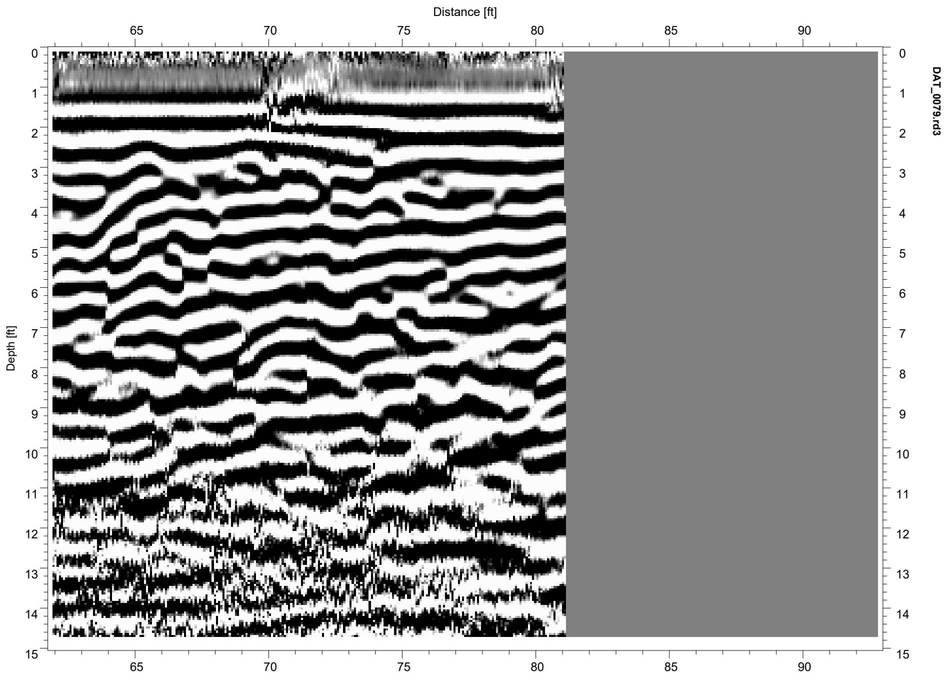

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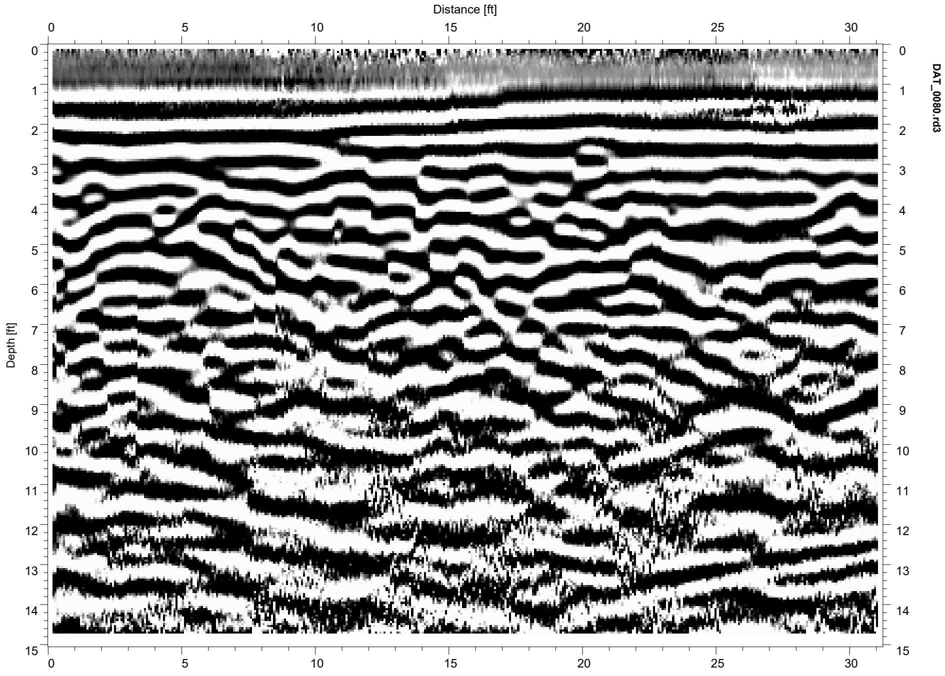

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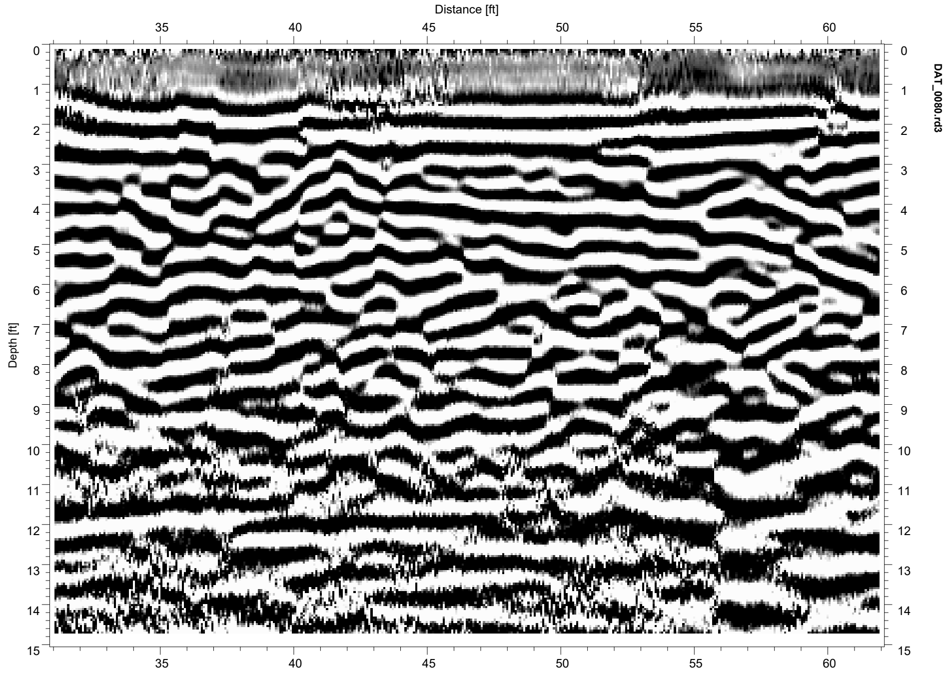

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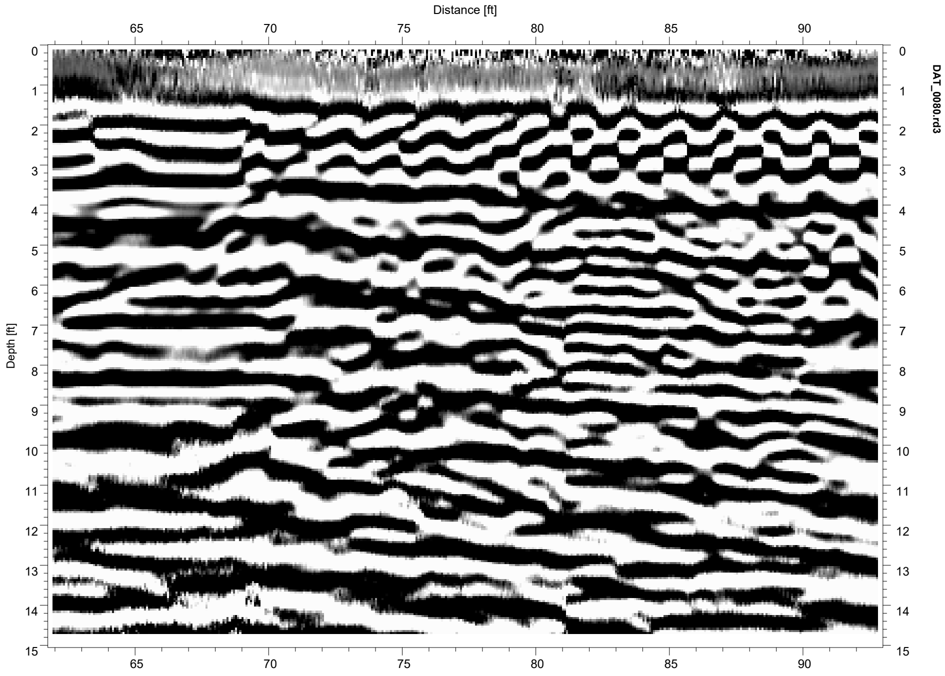

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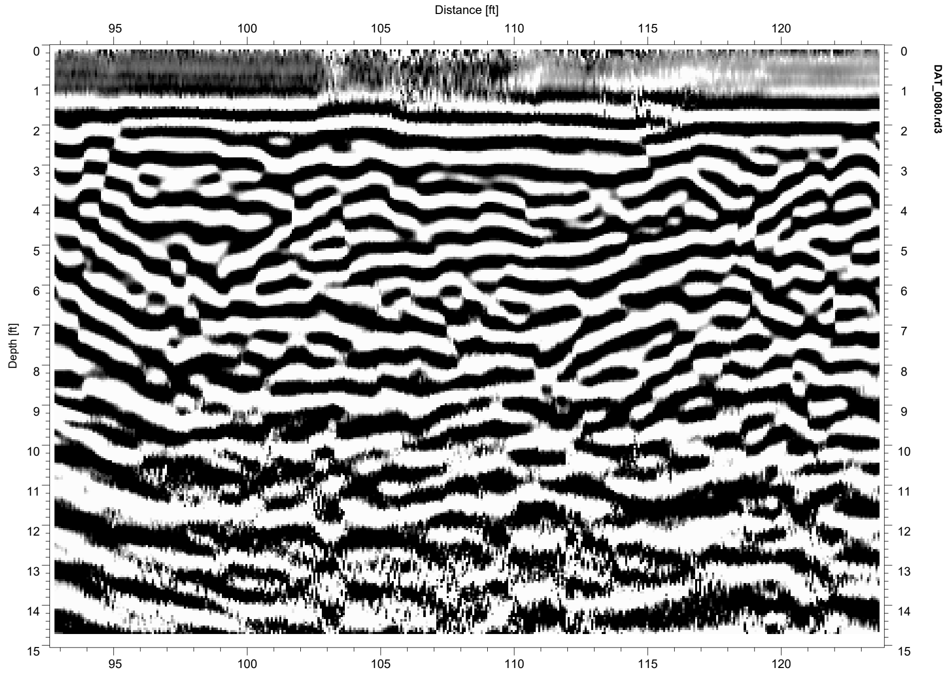

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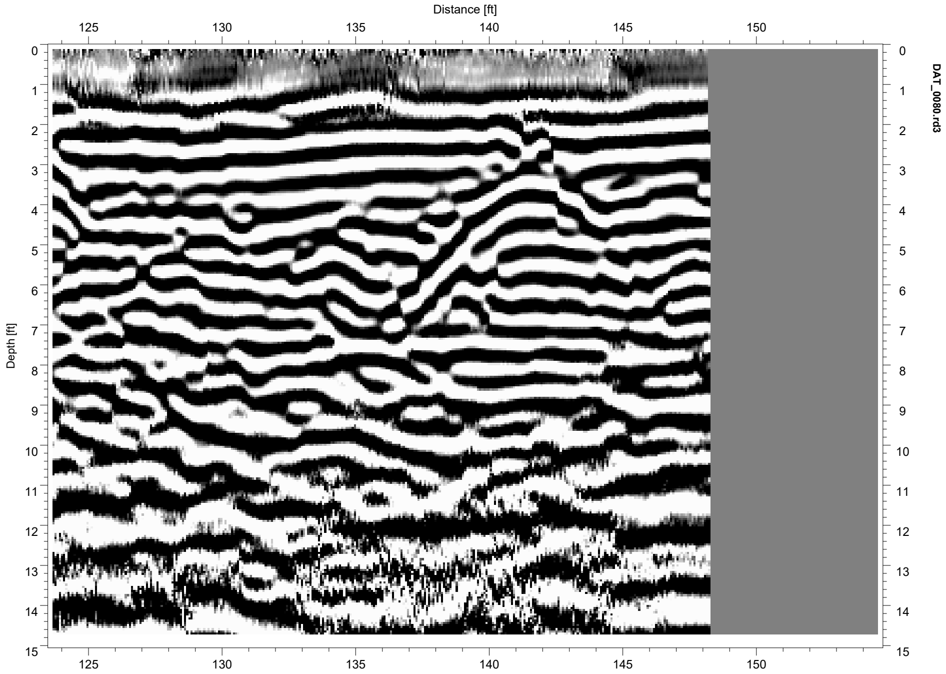

Forest Ave. Bridge

Latitude, Longitude: 40.73396585, -122.94235618

Date	5/28/2020, 1:26:25 PM
Design Code Reference Document	ASCE7-10
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	0.836	MCE_R ground motion. (for 0.2 second period)
S_1	0.39	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.975	Site-modified spectral acceleration value
S_{M1}	0.632	Site-modified spectral acceleration value
S_{DS}	0.65	Numeric seismic design value at 0.2 second SA
S_{D1}	0.421	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.165	Site amplification factor at 0.2 second
F_v	1.62	Site amplification factor at 1.0 second
PGA	0.363	MCE_G peak ground acceleration
F_{PGA}	1.137	Site amplification factor at PGA
PGA_M	0.413	Site modified peak ground acceleration
T_L	16	Long-period transition period in seconds
SsRT	0.836	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.89	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.39	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.426	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.94	Mapped value of the risk coefficient at short periods
C_{R1}	0.915	Mapped value of the risk coefficient at a period of 1 s

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Forest Ave. Bridge

Latitude, Longitude: 40.73396585, -122.94235618

Date	5/28/2020, 1:26:59 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.063	MCE_R ground motion. (for 0.2 second period)
S_1	0.519	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.142	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.762	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.075	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.491	MCE_G peak ground acceleration
F_{PGA}	1.109	Site amplification factor at PGA
PGA_M	0.544	Site modified peak ground acceleration
T_L	16	Long-period transition period in seconds
SsRT	1.063	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.186	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.621	Factored deterministic acceleration value. (0.2 second)
S1RT	0.519	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.593	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.867	Factored deterministic acceleration value. (1.0 second)
PGAd	0.764	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.896	Mapped value of the risk coefficient at short periods
C_{R1}	0.875	Mapped value of the risk coefficient at a period of 1 s

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FIVE COUNTIES SALMONID CONSERVATION PROGRAM

CONCEPTUAL PLANS FOR CONSTRUCTION OF

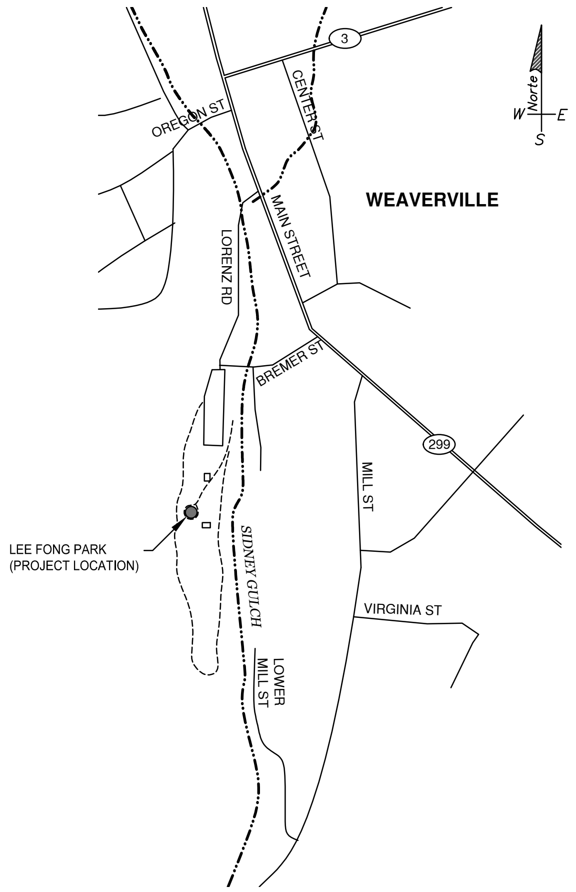
LOWER SIDNEY GULCH URBAN STREAM RESTORATION AT LEE FONG PARK, WEAVERVILLE, CALIFORNIA

MARCH 2019

VICINITY MAP
NOT TO SCALE

PREPARED FOR:

- FIVE COUNTIES SALMONID CONSERVATION PROGRAM (5C)
- NORTHWEST CALIFORNIA RESOURCE CONSERVATION & DEVELOPMENT COUNCIL
- COASTAL CONSERVANCY



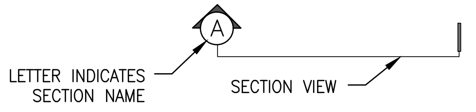
LOCATION MAP
NOT TO SCALE

SHEET INDEX	
No.	**SHEET TITLE**
1	TITLE AND COVER SHEET
2	CONCEPT PLAN - REACH 1 and 2
3	CONCEPT PLAN - REACH 3
4	CROSS SECTIONS

LEGEND,SYMBOLS AND ABBREVIATIONS

SYMBOLS

- ⇨ FLOW DIRECTION
- – - — - – PROPERTY LINE (APPROX.)
- ✿ (E) TREE
- ☒ TREE TO BE REMOVED
- ━ (N) PLACED WOOD



ABBREVIATIONS

APPROX/~	APPROXIMATELY
BF	BANKFULL
CL	CENTERLINE
CMP	CORRUGATED METAL PIPE
EG	EXISTING GROUND
EL	ELEVATION
(E)	EXISTING
FG	FINISHED GROUND, FINISHED GRADE
FT	FOOT OR FEET
IN	INCHES
MAX	MAXIMUM
MIN	MINIMUM
(N)	NEW
NTS	NOT TO SCALE
O.C.	ON CENTER
RSP	ROCK SLOPE PLACEMENT
STA	STATION
TBD	TO BE DETERMINED
TBM	TEMPORARY BENCHMARK
TYP	TYPICAL
W/	WITH
1.5:1	HORIZONTAL:VERTICAL SLOPE
%	PERCENT

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Five Counties Salmonid Conservation Program (5C)
P.O. Box 2571, Weaverville CA, 96093

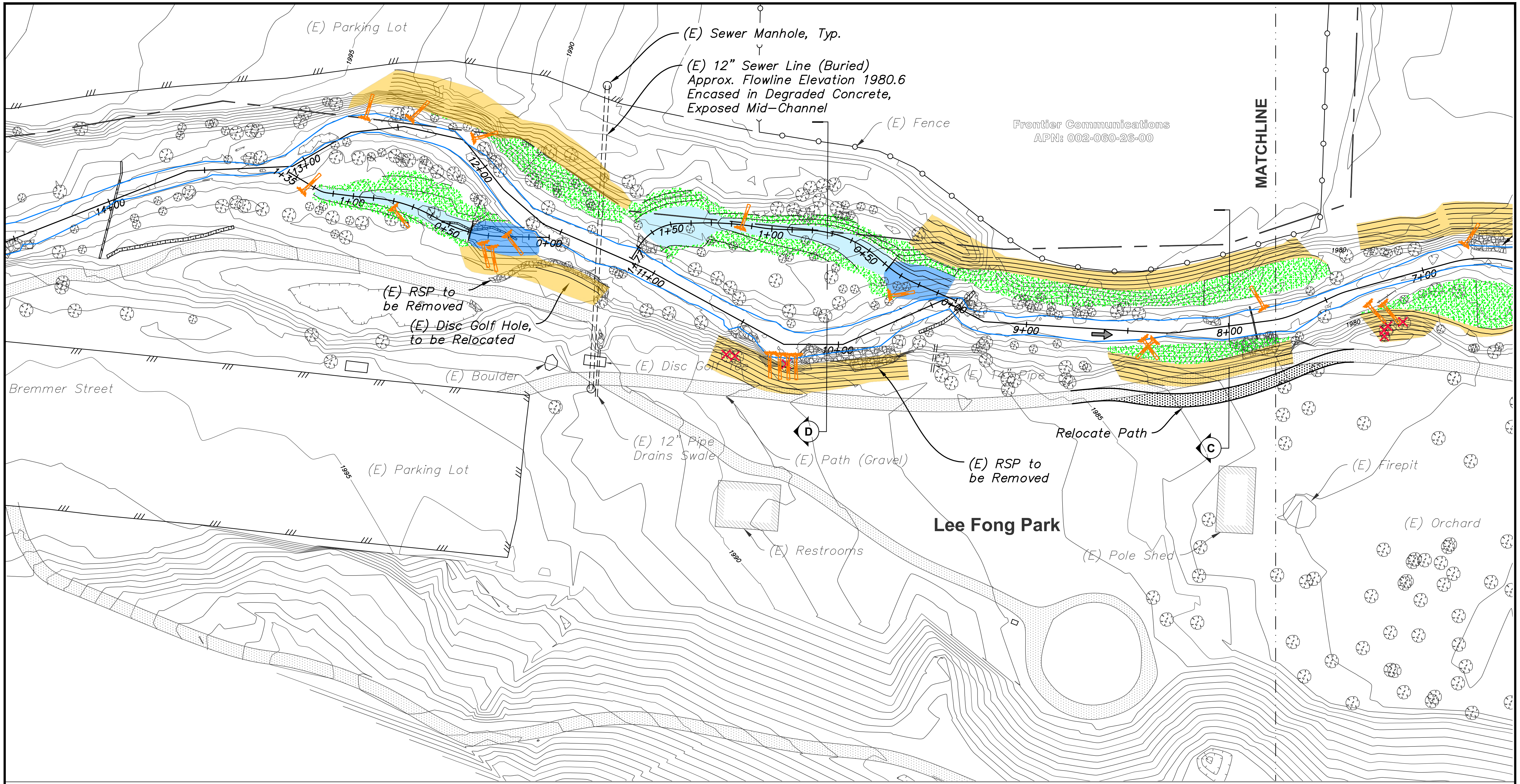
DRAFT

VERIFY SCALE
THIS BAR IS ONE INCH LONG AT FULL SCALE

FIVE COUNTIES SALMONID CONSERVATION PROGRAM
LOWER SIDNEY GULCH URBAN STREAM RESTORATION: PHASE 2 DESIGN

TITLE AND COVER SHEET

DATE
MARCH 2019
SUBMITTAL
30% DESIGN
DESIGN
ML / RS /AL
DRAWN
AL
SHEET
1 of 4



LEGEND

NEW CHANNEL	🟦
NEW FLOODPLAIN	🟩
NEW BANK GRADING	🟨
NEW SIDE CHANNEL	🟦
NEW ALCOVE	🟦
TREE TO BE REMOVED	☒

PLAN VIEW - REACH 1 & 2

SCALE: 0 25' 50'
GRAPHIC SCALE

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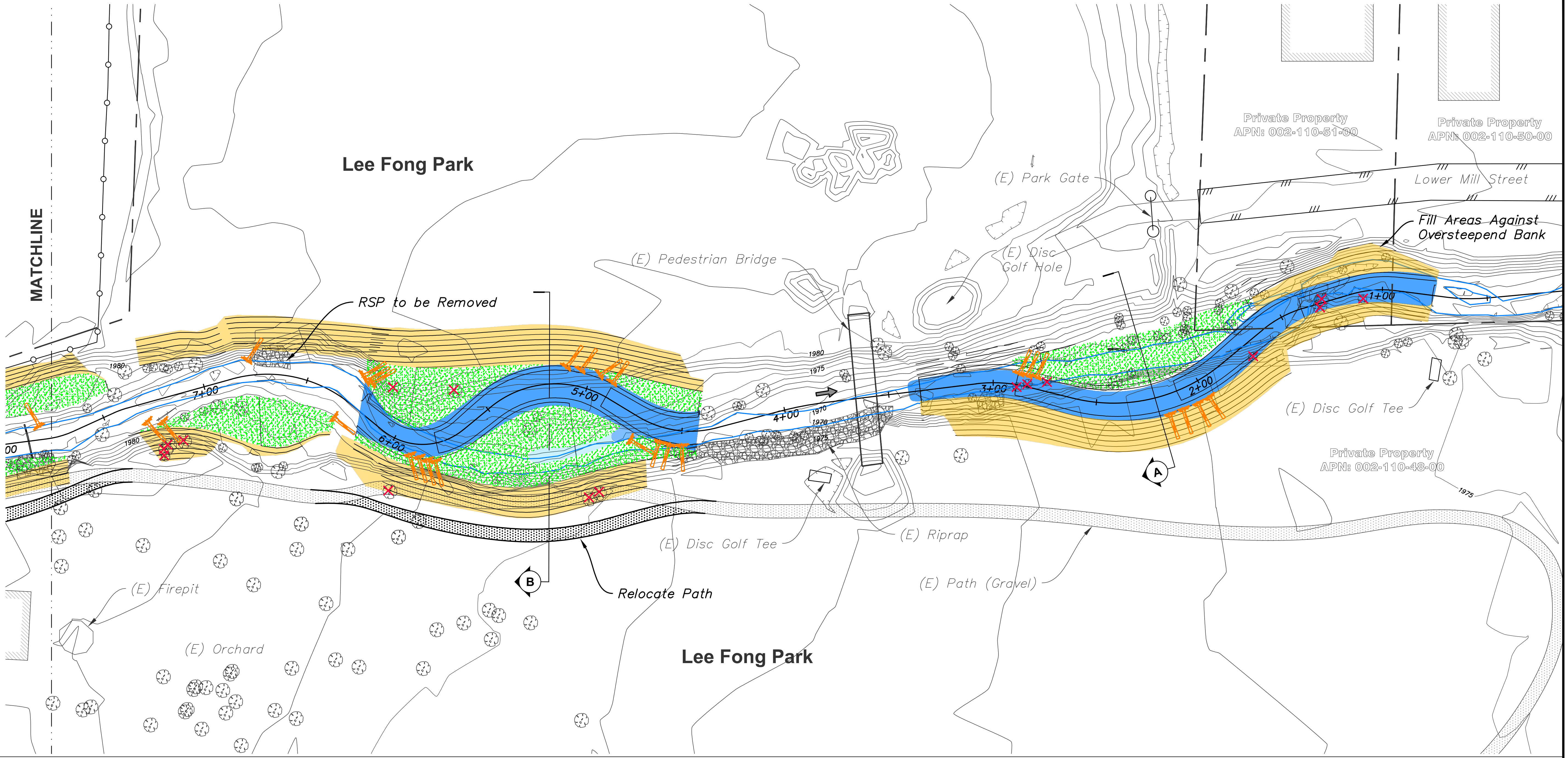
FIVE COUNTIES SALMONID CONSERVATION PROGRAM
LOWER SIDNEY GULCH URBAN STREAM
RESTORATION: PHASE 2 DESIGN

CONCEPT PLAN, REACH 1 AND 2

DATE
MARCH 2019
SUBMITTAL
30% DESIGN
DESIGN
ML / RS /AL
DRAWN
AL
SHEET
2 of 4

Q:\Sidney_Lee_Fong\7_CAD-GIS\Civil3D\Sheets\3_SG_PROPOSED_LAYOUT (Prelim).dwg



LEGEND

NEW CHANNEL	🟦
NEW FLOODPLAIN	🟩
NEW BANK GRADING	🟨
NEW SIDE CHANNEL	▭
NEW ALCOVE	🟦
TREE TO BE REMOVED	☒

PLAN VIEW - REACH 3

SCALE: 0 25' 50'
GRAPHIC SCALE

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FIVE COUNTIES SALMONID CONSERVATION PROGRAM
LOWER SIDNEY GULCH URBAN STREAM
RESTORATION: PHASE 2 DESIGN

CONCEPT PLAN, REACH 3

DATE
MARCH 2019
SUBMITTAL
30% DESIGN
DESIGN
ML / RS /AL
DRAWN
AL
SHEET
3 of 4

Q:\Sidney_Lee_Fong\7_CAD-GIS\Civil3D\Sheets\3_SG_PROPOSED_LAYOUT (Prelim).dwg

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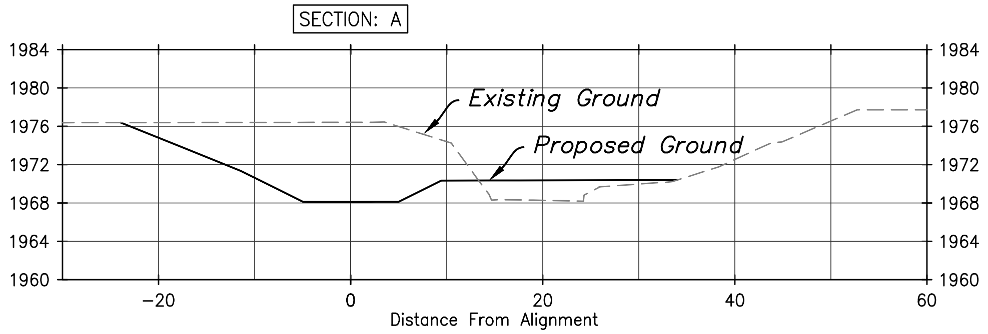

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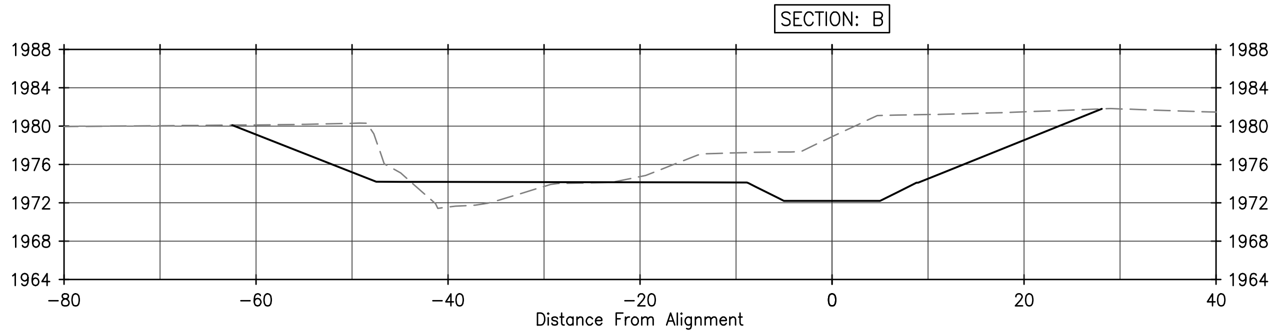

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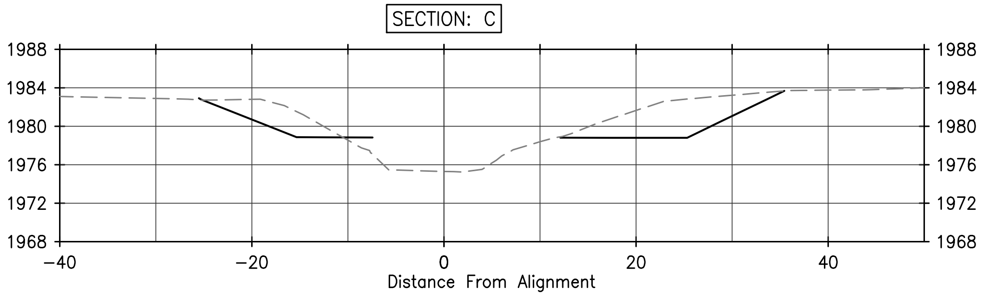

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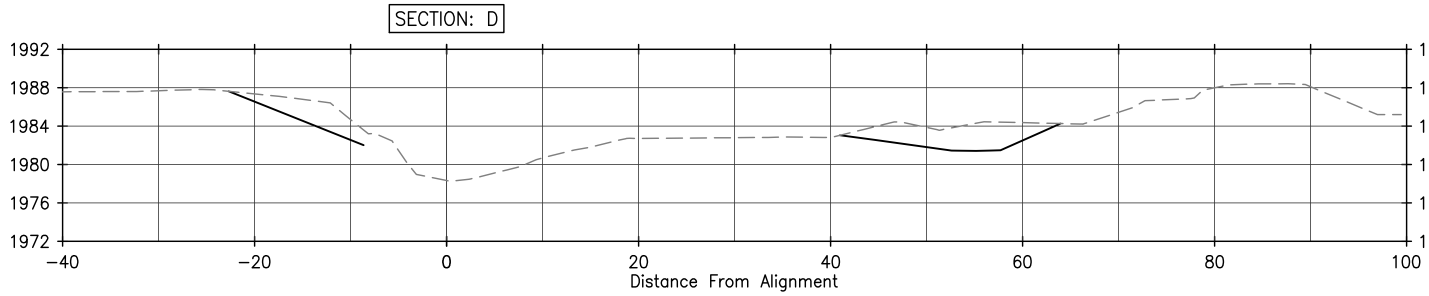

Sections Looking Upstream/Upstation

CROSS SECTIONS

HORIZONTAL AND VERTICAL SCALE: 0 10' 20' GRAPHIC SCALE

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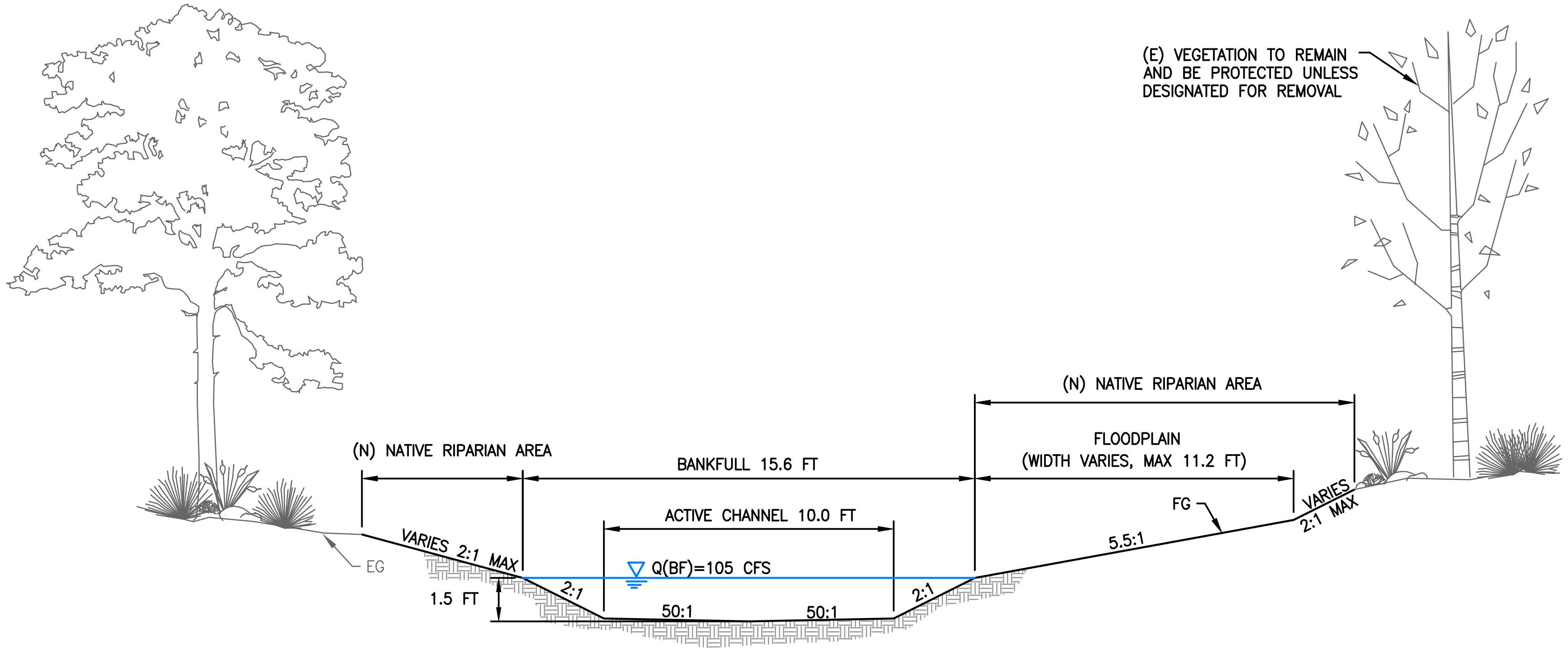
Conservation Program (5C)
P.O. Box 2571, Weaverville CA, 96093

VERIFY SCALE
THIS BAR IS
ONE INCH LONG
AT FULL SCALE

FIVE COUNTIES SALMONID CONSERVATION PROGRAM
LOWER SIDNEY GULCH URBAN STREAM
RESTORATION: PHASE 2 DESIGN

CROSS SECTIONS

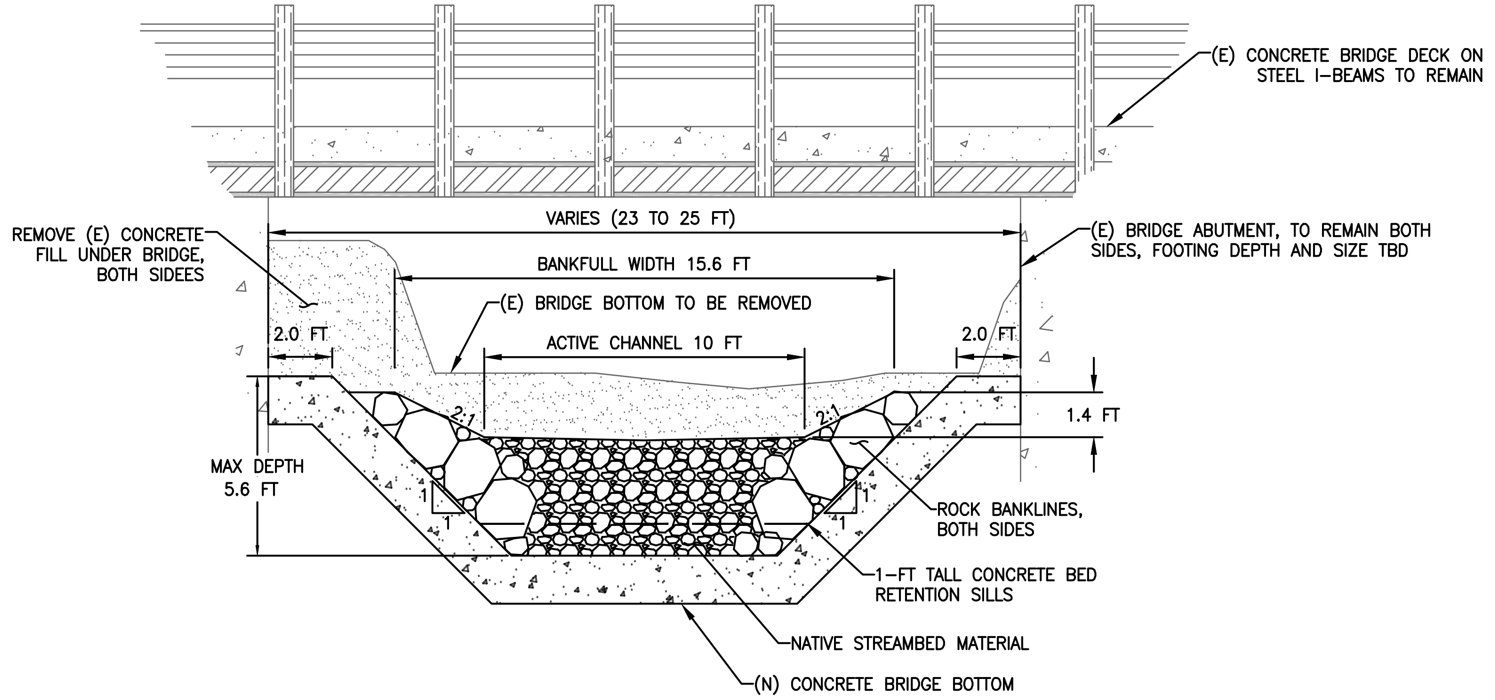
DATE
MARCH 2019
SUBMITTAL
30% DESIGN
DESIGN
ML / RS /AL
DRAWN
AL
SHEET
4 of 4



1 SIDNEY GULCH CHANNEL

TYPICAL GRADING SECTION

0 3 6
Feet



E SIDNEY GULCH AT FOREST AVE

DETAIL SECTION

0 3 6
Feet

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5C Program
Five Counties Salmonid Conservation Program

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NORTHWEST CALIFORNIA RESOURCE CONSERVATION & DEVELOPMENT COUNCIL
FISH PASSAGE DESIGN FOR THE SIDNEY GULCH USFS COMPOUND

TYPICAL SECTIONS

DATE
DECEMBER 2014
SUBMITTAL
CONCEPTUAL
DESIGN
RS, ML
DRAWN
AL, NN
SHEET
9 of 10