Appendix A
Dam Safety Evaluation

Appendix A, Dam Safety Evaluation, includes the following reports:

- **Water Supply Re-Allocation Study Dam Safety Evaluation Chatfield Dam, Littleton, CO**. This report is a geotechnical / structural dam safety evaluation of Chatfield Dam based on a potential permanent increase in the normal reservoir elevation by up to 12 feet.
- **Post-Liquefaction Stability Analyses.** This report presents the results of stability analyses performed on zones of the Chatfield Dam foundation that have been identified as susceptible to liquefaction.

Water Supply Re-Allocation Study
Dam Safety Evaluation
Chatfield Dam
Littleton, Colorado

US Army Corps of Engineers®

Water Supply Re-Allocation Study
Dam Safety Evaluation
Chatfield Dam - Littleton, Colorado

Water Supply Re-Allocation Study
Chatfield Dam
Geotechnical / Structural Dam Safety Evaluation

EXECUTIVE SUMMARY

Presented herein is a geotechnical / structural dam safety evaluation of Chatfield Dam based on a potential permanent increase in the normal reservoir elevation by up to 12 feet. It is emphasized that that this evaluation is based strictly on static loading scenarios and does not address seismic loading. It is vital to address various aspects of design and performance to assure that the proposed modifications do not impact the continued safe operation of the dam and do not pose dam safety concerns. The methodology utilized in this evaluation was to review design assumptions, evaluate instrument data compared to the design assumptions and evaluate historic performance of the project.

The requirement for a Phase I Seismic Study has been identified as a result of a Seismic Safety Review (SSR) of Chatfield Dam. A brief status of the seismic assessment is presented herein; however, seismic loading under new Corps of Engineers criteria (Phase I Study) has not been addressed in this report. The findings in this report are based strictly on normal static loading criteria.

Based strictly on a static evaluation of the project, no conditions have been identified that would prohibit adoption of the Re-Allocation Project. Installation of new piezometers located in the downstream fill, overburden and blanket drain are recommended as additional monitoring devices to assure continued safe operation of the project.

Although no dam safety concerns have been identified for the proposed reservoir loading, based on project performance and the instrumentation program, increased monitoring of the project will be required as part of the routine dam safety program to assure continued safe operation of the dam. This will include the development and implementation of a reservoir raise monitoring plan which would include additional inspection effort, instrumentation data acquisition and data analysis. The Project Surveillance Plan and Emergency Action Plan must also be updated as appropriate.

The analysis presented herein is based on a review of design assumptions, an evaluation of instrument data compared to the design assumptions and an evaluation of historic performance of the project. Any future dam safety concerns (seepage, slope stability, etc.) that may develop during/following the actual reservoir raise may have a direct bearing on the continued long term use of the re-located storage.

Water Supply Re-Allocation Study
Chatfield Dam
Geotechnical / Structural Dam Safety Evaluation

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Appendix F – Project Management Plan (PMP)

Water Supply Re-Allocation Study
Chatfield Dam
Geotechnical / Structural Dam Safety Evaluation

1.0 General. It has been proposed to raise the normal elevation of the Chatfield Reservoir by up to 12 feet, from El. 5432 feet to El. 5444 feet, for the purpose of water supply storage. The Water Re-allocation Study will actually evaluate three options, (1) no raise, (2) a five foot raise and (3) a twelve foot raise. The final adopted plan will not increase the maximum surcharge reservoir elevation. The historic maximum reservoir elevation at Chatfield Dam is 5447.58 feet. All elevations referenced hereinafter are NGVD 1929 Datum.

2.0 Purpose and Scope. The purpose of this report is to evaluate potential dam safety concerns based on a permanent increase in the reservoir elevation. The evaluation is based strictly on static loading; however, historic information on previous seismic evaluations will be presented. It is vital to address various aspects of design and performance to assure that the proposed modifications do not impact the continued safe operation of the dam and do not pose dam safety concerns. This evaluation will address the following areas of interest.

- ✓ Seismic
- ✓ Slope Protection
- ✓ Slope Stability
- ✓ Seepage
- ✓ Movement
- ✓ Structural

3.0 General Project Description. The Chatfield Dam and Reservoir Project is composed of a rolled earthfill dam, an ungated concrete spillway and stilling basin, an outlet works intake structure, two-barrel conduit and a stilling basin. A small flood detention dam, the Spring Gulch embankment, is located at the extreme right abutment of the main dam. General project drawings of the Chatfield Project are presented in Appendix A for information. Plate Nos. A-1 and A-2 present a general location plan and project plan. A list of pertinent data is as follows.

PERTINENT DATA

DRAINAGE AREA

Entire South Platte River Basin	24,030 square miles
Total above Chatfield Dam Site	3,018 square miles

RESERVOIR DATA (1991 Data)

	Elevation (feet m.s.l.)	Gross Storage (Acre-Feet)	Surface Area (Acres)
		Initial	
Maximum Surcharge Pool (Spillway Design Flood)	5521.6*	351,366	5,977
Flood Control Pool (Spillway Crest)	5500.0*	235,098	4,770
Multipurpose Pool	5432.0*	28,369	1,423
Streambed	5380.0*	0	0

* NGVD 1929

ELEVATIONS

Top of Dam*	5527.0 feet m.s.l.
Maximum Surcharge Pool (Spillway Design Flood)*	5521.6 feet m.s.l.
Spillway Crest*	5500.0 feet m.s.l.
Top of Multipurpose Pool* (Prior to Re-Allocation)	5432.0 feet m.s.l.
Top of Sediment Pool*	5426.0 feet m.s.l.

* NGVD 1929

DAM

Type	Rolled Earthfill
Maximum Height Above Riverbed	147 feet
Height Above Valley Floor	137 feet
Crest Length	13,136 feet
Fill Volume	17,255,100 cubic yards

OUTLET WORKS

Intake, Type	Tower with Access Bridge
Service Gates	
Number	2
Type	Hydraulically-Operated Slide
Size	5.5 feet X 13 feet
Emergency Gates	
Number	1
Type	Wheel-Wire Rope Hoist
Size	5.5 feet X 14.25 feet
Low-Flow Releases	
Gate Type	Gate-within-a-Gate
Size	2 feet X 2 feet
Auxiliary Conduit	
Size	72-inch diameter

Upstream Gate, Type	Butterfly
Number	1
Size	72-inch
Downstream Gate, Type	Butterfly
Number	2
Size	60-inch and 48-inch diameter
Bulkhead	
Number	1
Size	8 feet X 19.5 feet
Conduit Type	Twin Oblong
Maximum Height (inside dimension)	16.0 feet, each barrel
Maximum Width (inside dimension)	11.0 feet, each barrel
Length	756 feet
Invert Elevation, at Intake*	5385.0 feet m.s.l.
Invert Elevation, at Outlet*	5375.0 feet m.s.l.
Discharge Capacity (@ Elev. 5500.0)	8,300 cubic feet per second
Stilling Basin, type	Conventional Hydraulic Jump
Width	37 feet
Length	91 feet
Floor Elevation*	5358.0 feet m.s.l.
End Sill Elevation*	5361.0 feet m.s.l.

* NGVD 1929

SPILLWAY

Type	Ungated Chute
Bottom Width	390 feet
Crest Type	Ogee Weir
Crest Elevation*	5500.0 feet m.s.l.
Crest Length	500 feet
Stilling Basin	
Type	Conventional Hydraulic Jump
Width	390 feet
Length	154 feet
Floor Elevation*	5352.0 feet m.s.l.
End Sill Elevation*	5357.0 feet m.s.l.
Maximum Discharge	188,000 cfs @ Elev. 5521.6 feet m.s.l.*

* NGVD 1929

DOWNSTREAM CHANNEL

Capacity, maximum	5,000 c.f.s.
Width, minimum	100 feet
Length (approximate)	8 miles

WATER RIGHTS (Capacity)

	Total Decreed	D/S Requirements
Last Chance Ditch	43 c.f.s.	12 c.f.s.
Nevada Ditch	32 c.f.s.	32 c.f.s.
Denver Water Board		
City Ditch	86 c.f.s.	50 c.f.s.
Plum Creek Pump Station	34 c.f.s.	30 c.f.s.
Fish Hatchery	16 c.f.s.	16 c.f.s.

3.1 Embankment. Chatfield Dam is a rolled, zoned earthfill with a crest length of 13,136 feet, and a crest width of 30 feet. The maximum height of the embankment is 137 feet across the valley and 147 feet where it crosses the South Platte River. The embankment zoning consists of a symmetrical central impervious core with 1V on 3H side slopes; upstream and downstream random material shells; and a pervious inclined sand drain with continuous outlets adjacent to the downstream slope of the impervious core. The thickness of the pervious inclined drain is 20 feet in the valley sections and transitions to a 10 foot thickness in the abutments. An impervious cutoff trench excavated to bedrock through the pervious overburden materials joins the embankment core to provide a positive underseepage control. The outer portion of the downstream random zone includes a zone specifically for all Dawson Formation materials, which were excavated from the spillway, and outlet works excavations. The entire upstream face of the dam is protected with graded riprap. The grades (slopes) of the upstream and downstream slopes of the embankment are presented in Table Nos. 1 and 2.

Table No. 1 – Upstream Slope	
Slopes	**Elevations***
1V on 2.5H	5527.75 to 5493
1V on 5H	5493 to 5431
1V on 15H	5431 to 5420
1V on 3H	5420 to 5408
1V on 10H	5408 to Ground Surface

* NGVD 1929

Table No. 2 – Downstream Slope	
Slopes	**Elevations***
1V on 2.5H	5527.75 to 5493
1V on 5H	5493 to 5410
1V on 3H	5410 to Ground Surface

* NGVD 1929

Earthwork grading plans are presented on Plate Nos. A-3 thru A-7. Cross sections showing details of the embankment design are presented on Plate Nos. A-8 thru A-10. The toe drain profile and details are presented on Plate No. A-11.

3.2 Outlet Works. The outlet works is located near the left abutment of Chatfield Dam just to the right (south east) of the spillway. The outlet works discharge channel joins the spillway discharge pilot channel approximately 1500 feet below the toe of the dam. The outlet works structures consist of an intake structure, intake structure service bridge, twin oblong outlet works conduit and an energy dissipating drop structure and stilling basin. A plan and profile of the outlet works is presented on Plate Nos. A-12 and A-13.

3.2.1 Approach Channel. The approach channel to the outlet works intake has a length of approximately 700 feet at elevation 5385.0 feet m.s.l., which is also the intake invert elevation. A U-frame structure, 31.5 feet wide with vertical walls varying in height from 24 feet to 5 feet, lines the channel for a distance of 63 feet upstream from the intake structure. The remainder of the channel is excavated with 1V on 3H side slopes and is 10-feet wide. Details of the approach walls are presented on Plate No. A-14.

3.2.2 Intake Structure. The intake structure has three gated passageways that conduct water to the twin outlet works conduits. The two right passageways converge toward a conduit transition monolith in which the convergence to one passageway is completed. Each of these passageways has a service gate and emergency gate, which are controlled by hydraulic hoists. In each gate, a 2-foot by 2-foot auxiliary gate is provided to facilitate regulation of normal flows to the river. In the left passageway of the intake structure, a 6 foot diameter penstock, equipped with a butterfly valve near the upstream end, is provided to conduct releases to satisfy the downstream water rights. At the upstream end of the bellmouth entrance to each passageway are slots for bulkheads to facilitate maintenance of the gates and valves. Above the water passageways the intake is a rectangular, dry-well type structure with intermediate floors consisting of a hydraulic hoist chamber, bulkhead platform level, operating level and machine room level. An elevator and stairwells or embedded ladders furnish access between the floors. At the top of the intake structure, 142 feet above the invert, is a service deck that is accessible from the top of dam by a service bridge. A 10-foot high wall encloses the deck except for a 15 foot opening to provide access from the bridge. Concrete details of the intake structure are presented on Plate Nos. A-15 through A-17.

3.2.3 Trash Control. Vertical concrete trash beams with horizontal circular struts are provided for trash control at the inlet of the two water passageways on the right side (looking downstream). The clear openings are approximately 5-feet by 5-feet in dimension. These trash fenders prevent trees and large floating objects from entering the water passageways but will allow passage of smaller debris, which will normally go through the intake structure and conduit without damage.

3.2.4 Conduit. Each opening in the twin oblong conduit has a width of 12.0 feet and a height of 16.0 feet. A 5.5 foot radius, semi-circular arc on top and bottom connected by 5.0 feet straight vertical side walls, forms each opening. The discharge through the right passageway is controlled by the service gates for high pools and can maintain a maximum discharge of 5,000 c.f.s. The left passageway provides ample space for the penstock and walkway. The conduit and transition is 1,280 feet long, with a slope of 0.0076 feet/feet and invert elevations of 5385.0 feet m.s.l. and 5375.0 feet m.s.l. at the upstream and downstream ends, respectively. Conduit details are presented on Plate Nos. A-18 and A19.

3.2.5 Penstock. Three irrigation ditches were blocked off by construction of Chatfield Dam. To continue the water supply to these ditches, the left barrel of the conduit contains a 72 inch penstock, which has an independent inlet at the left side of the intake tower. The inlet includes a steel trashrack, bulkhead slots, bellmouth entrance, and a butterfly valve, which is normally operated fully opened. All bulkhead slots are the same width, thus a single bulkhead may be used for either of the two outlet works water passageways or for the penstock inlet. A manifold structure at the downstream end of penstock contains the gates and branch pipes, which distribute water to the ditches.

3.2.6 Stilling Basin. The stilling basin is a U-frame structure consisting of a drop section, 70 feet long and a level basin 88 foot long. Walls are vertical and are a maximum of 27 feet high in the level basin. The stilling basin is designed for a maximum discharge of 8,300 c.f.s. This corresponds to a pool elevation of 5,500 feet m.s.l. with both gates wide open. Details of the stilling basin, manifold structure and the water rights diversion pipes are presented on Plate Nos. A-20 through A-24.

3.2.7 Discharge Channel. The discharge channel connects the stilling basin with the spillway discharge pilot channel and the improved river channel. The bottom width is 100 feet. The side slopes are 1V on 3H and are riprapped for a distance of 200-feet from the stilling basin. Riprap is placed on the side slopes upstream from the end sill to guard against erosion.

3.2.8 Service Bridge. The intake service bridge is a 5-span, pre-cast, pretensioned box girder type bridge, 514'-6" long between the centerlines of bearings at the abutment and the intake tower. The bridge deck is 12'-0" wide between the 1'-3" high cast-in-place curbs on which 2'-1/2" high aluminum guard rails are mounted. The three box girders which support the bridge deck are tied together laterally by 1-1/4" diameter tensioned steel bars. The deck is constructed of cast-in-place concrete. The intermediate bridge supports are reinforced concrete bents with spread footings, approximately equally spaced. The abutment is a closed reinforced concrete structure that contains a vault for a transformer and other electrical devices. Bridge details are presented on Plate Nos. A-25 through A-29.

3.3 Spillway. The chute-type spillway is located in the left abutment of the dam. The spillway consists of an ungated ogee weir, 500 feet wide and 10 feet high above the top of the approach channel slab, a chute 838 feet in length and varying in width from 500 to 390 feet, and a stilling basin 390 feet wide and 154 feet long. The stilling basin floor is 148 feet below the crest of weir. The discharge channel has a bottom width of 550-feet with 1V on 5H side slopes. From the end sill, the channel floor gradually rises about 28-feet above the stilling basin floor and then slopes gently to the river. Sandstones, siltstones, and clay-shales of the Dawson Formation underlie the spillway area. Details of the spillway structure are presented on Plate Nos. A-30 through A-39.

3.3.1 Weir. The weir consists of nineteen 25 foot monoliths. A 12.5 foot section of weir is constructed integrally as a part of each abutment. The total weir length, at the centerline of the crest is an arc with a radius of 860.34 feet. The width of the weir parallel to the direction of flow is 50 feet. The crest of the weir is at elevation 5500.0 feet m.s.l. There is an 8 feet wide by 8 feet high gallery under the weir structure which houses foundation drains in the form of deep intercepting holes (PVC pipe lined) under the weir structure. These drains discharge into the gallery.

3.3.2 Abutments. On both the north and south extremities of the spillway weir, there is an abutment monolith 39.5 feet long measured normal to the direction of flow and 50 feet wide to match the upstream-downstream width of the weir. The length of each abutment monolith includes a 12.5 foot section of the weir. The spillway side face of each abutment above the crest of the weir is a vertical surface. Both abutments have a gallery entrance complete with stairs as well as ventilating and lighting systems. The abutments are identical in dimension, except they are opposite hand.

3.3.3 Approach Walls. The approach walls, which are identical in design on each side of the spillway, are cantilever type structures extending 150 feet upstream of the abutments. The walls are elliptical in alignment and were constructed in four monoliths. The upstream monolith is 25.67 feet long measured on the elliptical working line. The next three monoliths are each 52 feet long. All vertical joints contain waterstops.

3.3.4 Approach Slab. The 8-inch thick spillway approach slab extends 50-feet upstream from the face of the weir. A seat is provided for the slab at the weir on the toe of the approach walls and on the upstream cutoff structure. The joint between the slab and these structures is an expansion joint provided with waterstops. A 7 feet deep cutoff structure is provided at the upstream end of the approach slab to prevent entry of water under the slab during operation of the spillway. Beneath the slab is a drainage system that discharges into a manhole at the upstream end of slab.

3.3.5 Chute Walls. The cantilever type chute walls, which are identical in design on each side, extend downstream from the abutments a distance of 801 feet. There are waterstops in all the vertical joints, extending from the top of the wall stem to the waterstop between the base of the wall and the chute slab. The stem heights vary from about 30 feet downstream of the abutments to 14.5 feet high on the upstream 3 percent chute slope. On the 25-percent slope portion of the chute, the wall height is 13.5 feet with the exception of the downstream 134 feet where the stem varies from 13.5 feet to 47 feet in height.

3.3.6 Stilling Basin Walls. The stilling basin walls are cantilever type walls extending 154 feet downstream from the chute walls. There are waterstops included in each vertical joint from the top of the wall to the waterstop between the base of the wall and the stilling basin slab. The top of the wall is at elevation 5399 feet m.s.l. and the stem height above the stilling basin is 47 feet. The top 3 feet of the channel face of the stem is vertical while the lower portion has a 1V on 12H batter. The chute and stilling basin walls have backfill drains and an embedded drain collector, which discharges into the wall manholes.

3.3.7 Chute and Stilling Basin Slabs. The chute slab just downstream of the weir is level, elevation 5490 feet m.s.l., and is a circular segment with a length of 34.63 feet at the slab centerline. A circular curve 8 foot long forms the transition between the level area and a 3 percent slope. The slab then extends downstream on a 3 percent slope for a distance of 283.33 feet to elevation 5481.5 feet m.s.l. In this area, spillway width converges from 500 feet at the crest centerline to 390 feet at the downstream end of the 3 percent slope. The slab slope then changes to 25 percent for a distance of 518 feet to the stilling basin, elevation 5352 feet m.s.l. The stilling basin slab extends an additional 123 foot downstream at elevation 5352 feet m.s.l. to the seat on the end sill. The width of the 25- percent portion of the chute is 390 feet, and the width of the stilling basin slab is 382.67 feet. The transition between the two widths is between the beginning of the stilling basin and a point 176 feet upstream measured horizontally on the 25 percent slope. The slab is 1.5 feet thick from the weir to Station 4+85, from which the thickness increases to 4.0 feet at Station 8+03. The remainder of the chute slab is 4.0-feet thick. In the stilling basin, the slab is 6.0 feet thick under the baffles and 9.0 feet thick at the end sill. Beneath the slabs is a system of slab and foundation drains which discharge into manholes constructed in the walls on each side of the spillway.

3.3.8 Baffles. Two rows of baffles 5-feet high extend across the width of the stilling basin. The bottom of each baffle is recessed six inches into the slab and anchored by means of No. 9 bars hooked into the slab.

4.0 Construction History. The Chatfield Dam and Reservoir Project is one unit in the comprehensive plan for flood control of the South Platte River and its tributaries within Colorado, Wyoming, and Nebraska. The project was authorized for construction by the Flood Control Act of 1950.

Construction on the Chatfield Project began in August 1967, under contract No. DACW45-68-C-0023, Earthwork - Stage I, Valley Cutoff. A list of the major construction contracts for Chatfield Dam is given below.

- **Earthwork - Stage I**, Valley Cutoff, contract number DACW45-68-C-0023 was awarded to Johnson Bros. Highway and Heavy Contractors, Inc., and D. H. Blattner and Sons, Inc., of Litchfield, Minnesota.
- **Earthwork - Stage II**, contract number DACW45-68-C-0131 was awarded to Johnson Bros. Highway and Heavy Contractors, Inc., of Litchfield, Minnesota.
- **Earthwork - Stage III**, contract number DACW45-70-C-0095 was awarded to Holloway Construction Company, of Wixom, Michigan.
- **Outlet Works**, contract number DACW45-71-C-0058 was awarded to Wietz Company Inc., of Des Moines, Iowa.
- **Spillway Structure and Intake Service Bridge**, contract number DACW45-72-C-0088 was awarded to Hensel Phelps Construction Co., of Greeley, Colorado.

Closure of Chatfield Dam was completed on 1 August 1973 under the Stage II earthwork contract. The Earthwork Stage III contract was completed in 1974. The contract for the outlet works was issued and completed during the time of the Stage III earthwork contract. All of the major facilities at Chatfield Dam were essentially completed by April 1977.

5.0 Seismic Evaluation.

5.1 Preliminary Seismic Evaluation – 1985. A Preliminary Reconnaissance Report addressing seismicity was published in June 1985. The analysis indicated that the saturated pervious overburden materials in the upstream and downstream right abutment and valley section of Chatfield Dam would be susceptible to liquefaction during postulated earthquake shaking. The preliminary evaluation concluded that in the event that these potentially unstable silty sands, silty gravelly sands and silts should liquefy during this earthquake, the seismic stability of the embankment would be questionable. It was recommended therefore to proceed with additional analysis including additional drilling, sampling and testing.

5.2 Seismic Evaluation – November 1986 (D.M. PC-44). As a result of the finding of the Preliminary Seismic Evaluation (1985), a more in-depth evaluation was conducted. The findings of this analysis concluded that the embankment materials including the sand drain would be safe against liquification. The analysis also concluded that the overburden materials (upstream and downstream) would be safe from liquefaction except for an area along the upstream toe between Station 60+00 and 95+00 and between 400 feet upstream of the embankment centerline and the embankment toe. It was also concluded that even in light of the identified liquefaction zone, the embankment would remain stable during the postulated earthquake.

5.3 Seismic Safety Review - 2002. The Omaha District performed a Seismic Safety Review (SSR) for Chatfield Dam in Fiscal Year (FY) 2002. The SSR, including a Policy Compliance and Criteria Review (PCCR), were completed in FY05. This evaluation was performed in accordance with ER 1110-2-1806 (31 July 1995) "Earthquake Design and Evaluations for Civil Works Projects" and ER 1110-2-1155 (12 September 1997) "Dam Safety Assurance Program", Appendix B "Seismic Safety Evaluation Process for Embankment Dams and Foundations".

This SSR evaluated the adequacy of the previous seismic design evaluations presented in Design Memorandum No. PC-44 "South Platte River Basin, Chatfield Dam and Lake, Colorado, Seismic Evaluation", November 1986, to determine if changes in seismicity or analytical techniques would indicate that additional detailed evaluation was warranted.

Based on the findings of the SSR and the Independent Technical Review (ITR) comments, progression of this study to a Phase I Special Study was recommended to further evaluate the seismic hazards for Chatfield Dam. The SSR also recommended that the access bridge and the bridge piers be evaluated since the gates are dependent on the power feed that runs across the bridge.

5.4 Seismic Evaluation (Phase I). The scope of the Seismic Study was expanded to determine what impacts if any the higher pool elevations (up to elevation 5444 feet msl) have on seismic stability. The seismic analysis will be published under separate cover(s).

6.0 Slope Protection – The upstream slope protection material on the embankment consists of a 12" to 26" thick layer of dumped, quarried granitic gneiss riprap underlain by a 9" layer of spalls and a 6" layer of bedding. Typical cross sections of the dam along with slope protection sections and details are presented on the General Project Plates in Appendix A. The size and gradation of the material was designed using criteria developed by the Missouri River Division from wave tank tests.

The existing multipurpose pool elevation (El. 5432 feet m.s.l.) is slightly above the transition point from the 1V:15H rocked slopes, or in some cases natural ground, to the 1V:5H slopes as shown on Plate Nos. A-5 thru A-8 in Appendix A. The slope protection in this reach (1V:5H) consists of 20" of riprap placed on spalls and bedding. The new proposed elevation of the multipurpose pool (El. 5444 feet m.s.l.) will also be on the 1V:5H slope. Two distinct zones of horizontal riprap displacement have been identified at intermittent locations along the upstream slope during past inspections. The displacement is moderate (1 to 2 feet) and has been observed for several hundred feet along the shoreline. These displacement zones have been estimated to be between elevations 5434 and 5439 feet m.s.l. According to project personnel, the riprap was pushed up by ice-action during the winter of 1992-93. Although visual inspections indicate that 1 to 2 feet of displacement has occurred in some areas, no exposed spalls, bedding or embankment have been noted. A slight amount of displacement (1 foot or less) has also been noted near the existing normal pool line (approximate elevation

5430 to 5431 feet m.s.l.). The displaced stone has not been identified as a dam safety concern in recent Periodic or Annual Inspections. No remedial actions have been identified. These areas are monitored by project personnel on a routine basis. Raising the normal pool elevation by 5 to 12 feet should not have a direct bearing on the adequacy of the slope protection material; however, the slope protection material will continue to be monitored during routine dam safety inspections (monthly, annual, periodic,…). In addition to this, effort should be made to inspect existing areas of riprap displacement during low reservoir elevations.

7.0 Slope Stability.

7.1 General. Slope stability analyses were originally performed for three embankment sections during the design of Chatfield Dam: (1) embankment section at Station 95+00 where the embankment attains a maximum height of 137 feet and the depth of the alluvial material is approximately 55 feet; (2) the outlet works section, where the embankment attains a height of approximately 117 and alluvial material is 25 feet deep; and (3) the right valley embankment section at Station 68+50 where the embankment is approximately 131 feet high but where the Dawson Formation is at the ground surface for the downstream portion of the section and 30 to 40 feet below the surface under the upstream portion of the section. The analyses consisted of investigating four cases which simulate conditions of stress during the life of the structure. The cases were: (1) end of construction; (2) sudden drawdown; (3) partial pool and (4) steady state seepage. Detailed information on the various cases analyzed during design is presented in Appendix B, Plate Nos. B-1 thru B-7.

7.2 Method of Analysis. The sliding wedge method was used for the stability analyses. The factors of safety in the analyses were defined as the ratio of the available shear strength to the average necessary to maintain equilibrium.

7.3 Summary of Results. The results of the original stability analyses performed during the design stage of Chatfield Dam are presented in Table No. 3. The results of the stability analysis (original design) based on seismic loading are presented for information only. Slope stability from seismic loadings is fully addressed under cover(s).

Table No. 3 - Slope Stability Analysis Results *						
Case	Location	Critical Reservoir Elevation	Critical Factor of Safety			
			Normal		Earthquake	
			Actual	Req.**	Actual	Req.**
End of construction (u/s)	Sta. 95+00	NA	2.62	1.3	1.82	1.0
End of construction (d/s)	Sta. 95+00	NA	2.53	1.3	1.62	1.0
Partial pool (u/s)	Sta. 95+00	5460	1.49	1.5	1.04	1.0
Partial pool (u/s)	Sta. 104+35	5435-5450	1.46	1.5	0.90	1.0
Sudden drawdown (u/s)	Sta. 95+00	5500	1.33	1.2	0.92	1.0
Sudden drawdown (u/s)	Sta. 95+00	5521.6	1.23	1.0	na	na
Steady state seepage (d/s) - Conventional	Sta. 95+00	5500	1.43	1.5	.86	1.0
Steady state seepage (d/s) – At Rest pressures	Sta. 95+00	5335	1.13	-	-	-
Steady state seepage (d/s)	Sta. 104+35	5500	1.58	1.5	0.92	1.0
Steady state seepage (d/s)	Sta. 68+50	5500	1.62	1.5	0.94	1.0

* Documented in the Embankment Criteria and Performance Report, April 1980
** Based on requirements presented in EM 1110-2-1902

End of Construction, Station 95+00 - Plate No. B-1 presents the "end of construction case" (upstream & downstream) at Station 95+00. Adequate (above minimum required) factors of safety were obtained for these cases.

Partial Pool, Station 95+00 - Plate No. B-2 presents the upstream slope partial pool analysis at Station 95+00. The analysis produced a minimum (critical) factor of safety of 1.49 at pool elevation 5460 feet m.s.l. with the failure plane in the random fill. The minimum required factor of safety is 1.50. According to the analysis, the factor of safety increased both above and below this critical pool elevation (5460 feet m.s.l.) as shown on Plate No. B-2. According to the analysis, the factor of safety would be at or above the minimum required 1.5 value for a pool elevation of 5444 feet m.s.l.

Sudden Drawdown, Station 95+00 - Plate No. B-3 presents the results of the sudden drawdown analysis of the upstream slope at Station 95+00. Sudden draw-down stability (upstream slope) was evaluated for both the spillway pool (El. 5500 feet m.s.l.) and the maximum surcharge pool (El. 5521.6 feet m.s.l.). The minimum

calculated factor of safety starting with a spillway pool (El.5500 feet m.s.l.) was 1.33 while the factor of safety at a surcharge pool (El. 5521.6 feet m.s.l.) was 1.23. The required factors of safety in these cases are 1.2 for pools starting at the spillway crest (5500 feet m.s.l.) and 1.0 for the maximum pool elevation (5521.6 feet m.s.l.). The analysis was performed using a conservation pool of El. 5426 feet m.s.l. whereas the actual conservation pool elevation (prior to a pool raise associated with the Re-Allocation Study) is El. 5432 feet m.s.l., six feet above what was used. This would make the analysis more conservative. The re-allocation study assumes an increase in the conservation pool elevation of up to 12 feet. This would be 18 feet above the water level used in the original design analysis (El. 5426 feet m.s.l.). Based on the conservative strengths assumed in design (as documented in the Embankment Criteria & Performance Report and discussed hereinafter), the factor of safety of 1.23 (1.0 required as minimum) at the lower conservation pool of El. 5426 feet m.s.l. and the increase in the conservation pool elevation, it is felt that the dam would have an adequate factor of safety for sudden drawdown.

Historic analysis (design stage) was based on USACE guidance available at that time and did not consider transient seepage conditions. Based on the conservative strengths assumed in design (as documented in the Embankment Criteria & Performance Report and other documents) higher conservation pool, and design calculated factor of safety of 1.23 (1.0 required as minimum), additional transient analysis is not considered necessary.

Steady State Seepage, Station 68+50, 95+00 & 104+35, - The downstream slopes of the embankment were analyzed under a steady state condition at Stations 95+00, 104+35 and 68+50 using the spillway crest pool elevation (El. 5500 feet m.s.l.). Plate No. B-4 presents the steady state seepage case at Station 95+00. Plate No. B-5 presents the steady state seepage case at Station 104+35. Plate No. B-6 presents the steady state seepage case for Station 68+50. These sections were considered to be the most critical of all the sections studied. Saturation through the embankment was assumed to be from the spillway crest pool (El. 5500 feet m.s.l.). The maximum pool elevation (El. 5521.6 feet m.s.l.) was not used in the analysis since it was considered that this pool elevation would not be maintained long enough to produce seepage equilibrium through the embankment. The downstream piezometric level was assumed to be at the top of the pervious blanket drain (elevation 5400 feet m.s.l. at Stations 95+00 & 68+50 and elevation 5420 feet m.s.l. at Station 104+35). There appears to be a conflict between various documents as to the exact elevation and thickness of the drain. It is documented in the Embankment Criteria and Performance Report that the thickness of the drain is 20 feet in the valley and 10 feet at the abutments. The drawings show 10 feet at Station 62+00 and 20 feet at Station 70+00 whereas the cross section used for the steady state seepage stability analysis (Embankment Criteria and Performance Report) at Station 68+50 shows a 10 foot drain with top elevation of 5405 feet m.s.l. It would be advantageous to verify the exact thickness of the drain.

Station 95+00 produced the lowest factor of safety for steady state seepage conditions. As shown in Table No. 3, the computed factor of safety at Station 95+00 was 1.43 for normal conditions with the critical failure plane in the Dawson Formation. This value is 0.07 below the required minimum factor of safety (1.50). The proposed Re-Allocation project would result in a maximum new normal pool elevation of 5444 feet m.s.l. This is 56 feet lower than the reservoir elevation (5500 feet m.s.l.) analyzed. The Re-Allocation will not increase the reservoir elevation analyzed in this case (5500 feet m.s.l.) and therefore the new normal pool elevation would not be expected to reduce the factor of safety calculated during design. The factors of safety at Stations 104+35 and 68+50 were 1.58 and 1.62 respectively as presented in Table No. 3 and on Plate Nos. B-5 and B-6.

Non-circular, block failure surfaces were analyzed at Station 95+00 and 68+50 with Spencer's method utilizing the 2007 version of SLOPE/W, developed by Geo-Slope International, Ltd. For the re-analysis of the steady state seepage cases, the critical slide plane and peak effective strengths were used as presented in the Embankment Criteria and Performance Report (CENWO, 1980). The intent of these re-analyses was to compare the current methodology (Spencer's method) with the method used for the original design. The results of the re-analysis of the steady seepage cases at Station 95+00 and Station 68+50 are presented in Table No. 4. These results indicate the factors of safety determined with Spencer's method exceed those factors of safety determined during the original design.

Table No. 4

Station	Factor of Safety		
	Original	Re-Analysis	
		Janbu's Method	Spencer's Method
95+00	1.43	1.46	1.74
68+50	1.62	1.57	1.84

The difference in the Factor of Safety between the original analyses and the re-analyses using Spencer's method is due to the side force assumptions and statics of each method.

Upstream Slope Partial Pool, Station 104+35 - The upstream slope partial pool analysis (normal loading) at Station 104+35 produced a minimum factor of safety of 1.456 @ pool elevation 5435 feet m.s.l. with the failure plane in the Dawson Formation, as presented on Plate B-5. The required minimum factor of safety is 1.50. The factor of safety increased slightly both above and below this critical elevation (5435 feet m.s.l.) as shown on Plate No. B-5. According to the information presented on Plate No. B-5, the factor of safety at the proposed new normal pool elevation (5444 feet m.s.l.) would also be approximately 1.46. It is pointed out that the current normal pool elevation is 5432 feet m.s.l. and the reservoir has historically operated approximately between elevations 5428 feet m.s.l. and 5447 feet m.s.l.

Although factors of safety slightly lower than that required by Corps criteria were obtained during the original design, the analyses were considered “as adequate” because of the exceptionally conservative adopted strengths and other assumptions in the analyses. The adopted design strength of the embankment material was based on primarily the lowest strength material of all the types placed in the embankment. The bedrock was assumed to be homogeneous in strength with no allowances made for cross bed shear and the sandy and silty phases of the Dawson Formation. The value of the adopted strength of the Dawson Formation was taken as an average between the residual shear strength test values (approximately 8 degrees) and the average peak strength of 24 degrees. The resulting adopted phi value of 15 degrees for the Dawson Shale, as shown on Plate No. B-7, was considered to be very conservative.

Additional slope stability analyses including soil testing were performed for the section of embankment at the outlet works as presented in "Supplement to Design Memorandum No. PC-24, Embankment and Excavation, December 1970." The general consensus in the August 1968 Board of Consultants meeting was that the conventional wedge analysis may not be applicable where failure is assumed in the Dawson Formation. The Board discussed the potential for strain incompatibility between the Dawson Formation and the embankment materials. The new test results fell within the range of the previous test data. The additional analyses involved no change in shear strength but took into account some or all of the following: (1) apparent soft seams in the Dawson Formation and their depth relative to the slide planes; (2) earth pressures at rest; (3) seepage pressures in the foundation shale; (4) pore pressures recorded at Fort Peck Dam which were translated to these studies; and (5) excavations of the intake and stilling basin included in some analyses. Specific cases analyzed are as follows:

End of Construction – Sta. 103+34: These studies were made assuming the following.

- Pore pressure responses in the Dawson Formation varying from 0 to 100%.
- Driving forces computed from earth pressure theory using an at-rest pressure coefficient of 0.5.
- Strengths assumed along failure plane: phi = 15 degrees & cohesion = 0

Depending on the specific assumptions in the analysis, the computed upstream slope factors of safety ranged from 1.77 to 0.81. The downstream factors of safety were approximately 0.2 higher than calculated for the upstream slope.

Steady Seepage Case – Sta. 103+34: All design strengths were the same as used in the original analysis presented in the basic design memorandum. The factors of safety computed ranged from 1.22 to 1.24. In this case the factors of safety were considered ….”satisfactory due to the relatively low adopted shear strength and other maximum conditions assumed in the analysis”[1].

[1] Supplement to Design Memorandum No. PC-24, Embankment and Excavation, December 1970.

Partial Pool Case – Sta. 103+34: All design strengths were the same as used in the original analysis presented in the basic design memorandum. For three pool levels analyzed the critical pool was at elevation 5450 feet m.s.l. with a factor of safety of 1.34. In the original stability analysis, a factor of safety of 1.46 was computed for pool elevations 5435 feet m.s.l. and 5450 feet m.s.l. "A factor of safety of 1.5 usually is required for conventional type analyses; however, the lower factor of safety appears justified here due to the use of the conservative strength assumptions"[1].

Sudden Drawdown Analysis – Sta. 103+34: For the case of drawdown from pool elevation 5500 feet m.s.l. to the minimum pool elevation of 5426 feet m.s.l. using various assumptions, a factor of safety of from 0.84 to 1.0 was obtained. It was concluded that .. "It is extremely unlikely that all of the above conditions assumed in the analyses would ever be met, that is, the assumptions used have been conservatively chosen, and as such the resulting factors of safety reflect those assumptions".[1]

Steady Seepage – Sta. 95+00: A revised analysis of the downstream slopes of the embankment at Station 95+00 was performed using at-rest pressures. The resulting factor of safety for the "At-Rest Pressure Analysis" was 1.13. "For this condition, with all of the maximum conditions imposed in the analysis, maintaining equilibrium is considered satisfactory".[2]

Results of the additional studies gave factors of safety lower than those previously obtained; in some cases below equilibrium conditions (less than 1); however, it was recognized that the many different assumptions were, as a whole, extremely conservative and as a result, the factors of safety so obtained reflected those assumptions. No definite conclusions/recommendations were identified in the Supplement to Design Memorandum PC-24 other than the use of conservative assumptions; however, it was stated that this subject would be further addressed in the Board of Consultants meeting scheduled for the fall of 1971. There was very little information documented about embankment stability and/or test results in the minutes of this meeting; however, no stability concerns were identified. Author Casagrande stated in part…"On this stability analysis, I have the impression that the assumptions are certainly on the safe side…."[3]

A discussion of the current and projected piezometric levels as compared to what was assumed in the design phase (stability analysis) is presented hereinafter.

<u>7.5 Instrumentation Review.</u> A piezometer location plan is presented in Appendic C, Plate No. C-1. The Chatfield Dam piezometer data is presented on Plate Nos. C-2 thru C-10. Plate Nos. C-11 thru C-14 presents various cross sections exhibiting piezometeric data. Plate Nos. C-15 thru C-35 present specific detailed piezometer plots. These plots were developed to identify any potential reflection of fluctuations in

[2] Design Memorandum No. PC-24, Embankment & Excavation, Dec 1968
[3] Meeting of Board of Consultants, Chatfield Dam, 16 Nov 1971

the reservoir level and ultimately the effect of an increased pool loading condition (piezometeric data) if any.

The upstream piezometric levels including data from piezometers located in the core are fairly responsive to the reservoir elevation and in some cases there is minimal headloss. Based on this piezometer data along with data projections as discussed hereinafter, it appears that the piezometric levels in both the upstream area and the core are/will be very similar to those assumed in the design (stability analysis).

The downstream piezometers are affected by the pool to a much lesser degree than the upstream and core piezometers as expected. The piezometer levels in the downstream bedrock (Dawson Formation) are currently at or below the elevation assumed in the steady state stability analysis at Stations 68+50, 95+00 and 104+35. Based on the relationship with pool fluctuations, it appears that there are potentially three critical downstream piezometers that include instruments numbers 504A, 505B, and 561. Piezometer 504A is a pneumatic pressure cell located at Station 68+90, 195 feet downstream. Piezometer 505B is a pneumatic pressure cell located at Station 68+90, approximately 400 feet downstream. Piezometer 561 is an open tube device located at Station 93+00, approximately 300 feet downstream. Data plots for these instruments are presented on Plate Nos. C-23, C-23A, C-25, C-25A, C-25 and C-28A.

These three piezometers monitor pressures in the downstream Dawson Formation and currently exhibit the highest piezometric levels of the downstream bedrock instruments, approximately elevation 5400 feet m.s.l. (piezometric level used in stability analysis). These instruments also exhibit minor fluctuations which are assumed to be related to the reservoir level.

A review of the data from Piezometer 504A reveals a slight decrease in level from approximately elevation 5408 feet m.s.l. in 1979 to the current level of approximately 5400 feet m.s.l. as shown on Plate Nos. C-23 and C-23A. There does appear to be a slight reflection of the reservoir elevation in the piezometer data, in particular in June 1995 during a high pool elevation. The tip elevation of Piezometer 504A; however, is 5338.3 feet m.s.l. This is approximately 18 feet above the elevation of the critical “failure plane” as documented on Plate No. B-6.

A review of the data from Piezometer 505B reveals a slight increase in level from approximately elevation 5396 feet m.s.l. in 1975 to the level of approximately 5400 feet m.s.l.in 1985 as shown on Plate Nos. C-25 and C-25A. From 1985 to the present the piezometric level has remained fairly constant at the approximate elevation 5400 feet m.s.l.. There does not appear to be a substantial reflection of the reservoir elevation in the piezometer data. The tip elevation of Piezometer 505B; however, is 5386.6 feet m.s.l. This is approximately 66 feet above the elevation of the critical “failure plane” as documented on Plate No. B-6.

A review of the data from Piezometer 561 reveals a definite decrease in level from approximately elevation 5415 feet m.s.l. in 1979 to the current level of approximately

5400 feet m.s.l. as shown on Plate Nos. C-28 and C-28A. There does appear to be a substantial reflection of the reservoir elevation in the piezometer data, primarily in 1995. The tip elevation of Piezometer 561 is 5297 feet m.s.l. The sensing zone is one foot below the tip to 3 feet above the tip. This is approximately the elevation of the assumed "failure plane" used in the stability analysis at Station 95+00.

The piezometric levels in these three instruments are currently at or below the piezometric level assumed in design (5400 feet m.s.l.); however, minor reflections of reservoir fluctuations have been observed primarily in Piezometer 561 in 1995. Additional bedrock piezometers with tip elevations in the "critical failure plane" would be warranted to better monitor the effects of higher pools. This would also enable the determination of the elevation and depth of the drain.

There are no piezometers located in the drain and very few in the downstream overburden. Additional piezometers in both the drain and downstream overburden would be warranted if a pool raise were to be implemented. Preliminary locations of proposed new piezometers are Stations 69+00, 81+00, 93+00 and 102+00. Additional piezometers would not only provide needed piezometric data during high pool elevations but would also enable the determination of the exact elevation and thickness of the drain.

The existing piezometer data is at or below that which was assumed in design (slope stability); however, two piezometers (504A & 561) in the Dawson Formation have exhibited a reflection to fluctuation in the reservoir elevation. Based on historic records, primarily during the record pool in 1995 these piezometers reflected the pool change by 15 to 20%. Piezometer 504A is a pneumatic cell located at Station 68+90, Range 195 d/s. Piezometer 561 is an open tube device located at Station 92+90, Range 300 d/s. Both of these instruments monitor pressure in the Dawson Formation. A piezometric level of 5405 feet m.s.l. was assumed in the stability analysis at Station 68+50. Piezometer 504A is currently reading approximately 5401 feet m.s.l. If the normal pool elevation was increased by 12 feet, this instrument could, based on historic fluctuations, increase to the approximate elevation of 5403 feet m.s.l. This is still below what was assumed in the design. It is also stressed that the factor of safety in the original analysis at Station 68+50 was 1.62. Piezometer 561 located at Station 92+90 is currently reading the approximate elevation of 5398 feet m.s.l. A piezometric level of 5400 feet m.s.l. was assumed in the stability analysis at Station 95+00. The current piezometric level observed by Piezometer 561 is approximately 5398 feet. m.s.l. If the normal pool elevation was increased by 12 feet, this instrument could, based on historic fluctuations, increase to the approximate elevation of 5400 feet m.s.l., the elevation used in the design. A potential concern is for development of pressures that exceed those used in the design analysis at a reservoir elevation of 5500 feet m.s.l. The piezometric level in both Piezometer 504A and 561 would be expected to exceed the level used in design based on past performance. Piezometeric levels exhibited by these instruments do not pose a concern related to the Re-Allocation Project; however, this concern should be pursued as part of the routine dam safety program.

A review of the embankment movement data (inclinometers and survey points) does not reveal any areas of instability or potential instability. Movement data (surveys) along with inclinometer data is presented in Appendix D as discussed hereinafter. No relationship has been identified between movement (potential instability) and thrust of the pool; however, the maximum pool elevation to date has been approximately 5447.58 feet m.s.l. and this elevation was maintained for a relatively short time period.

8.0 Seepage Control.

8.1 Embankment Seepage.

8.1.1 General. The embankment was designed with a central symmetrical impervious core with 3V on 1H side slopes to provide an effective barrier against through seepage. A pervious inclined sand drain with a blanket outlet was placed adjacent to the downstream slope of the impervious core to intercept and dissipate any seepage through the embankment. According to the Embankment Performance and Criteria Report the thickness of the drain in the valley is 20 feet and transitions to a 10 foot thickness in the abutments.

8.1.2 Instrumentation Review. A review of the embankment (core) piezometer data, as presented in Appendix C, revealed a slight influence of the reservoir on piezometric levels for some of the instruments located in the impervious core, in particular the following instruments.

- **Piezometer No. 502A.** Piezometer 502A is a hydrostatic pressure cell located at Station 68+90. It is not know for sure if the cell is located in the bedrock, at the interface of the bedrock and the core or completely in the core. The observed piezometric level has fluctuated only slightly over the years, plotting at the approximate elevation of 5417. Since 1980 there has been; however, a very slight but steady overall decline in the piezometric level exhibited by this instrument.

- **Piezometer No. 502B.** Piezometer 502B is a hydrostatic pressure cell located in the core at Station 68+90. The observed piezometric level has fluctuated only slightly over the years, plotting at the approximate elevation of the downstream blanket drain; however, the piezometric level exhibited by this instrument increased approximately 4 feet in early 2005. The reason for this is unknown; however, this increase is not consistent with reservoir fluctuations. Pool influence is also not apparent in historical data. See Plate No.C-15, Appendix C for a detailed plot.

- **Piezometer No. 507C**. Piezometer 507C is a hydroststic pressure cell located in the core at Station 81+20. The readings have been somewhat erratic; however, there is a general slight relationship with fluctuations in the pool elevation. See Plate No.C-16, Appendix C for a detailed plot.

- **Piezometer No. PZ95-03.** Piezometer No. PZ95-03 is an open tube located in the core at Station 81+20. The readings are approximately 10 feet below the reservoir elevation and 10 feet above the elevation of the downstream blanket drain. There is a definite relationship with fluctuations in the pool elevation. See Plate No.C-17, Appendix C for a detailed plot.

- **Piezometer No. 83+00/CTR**. Piezometer 83+00/CTR is an open tube device located in the core at Station 83+00. This instrument fluctuates only very slightly if at all with the elevation of the reservoir with the readings at or below the bottom of the blanket drain located downstream of the core. See Plate No.C-18, Appendix C for a detailed plot.

- **Piezometer No. 102+00/CTR.** Piezometer No. 102+00/CTR is an open tube device located in the core at Station 102+54. The readings fluctuate very slightly with a general decline over the past 10 years. The elevation of the readings is at or slightly above the top of the downstream blanket drain. No definite relationship with fluctuations in pool elevation is apparent. See Plate No. C-19, Appendix C for a detailed plot.

- **Piezometer No. 519C**. Piezometer No. 519C is a hydrostatic pressure cell located in the core at Station 102+54. Up to approximately 7 feet of fluctuation has been observed in this instrument since 1998. There has been a general increase (3'-5') over the past 7 years. The elevation of the readings is approximately 15' above the top of the downstream blanket drain. A slight relationship with fluctuations in pool elevation is apparent; however, a current investigation has revealed that one of the lines is plugged making the data extremely questionable. See Plate No. C-20, Appendix C for a detailed plot.

Piezometer 512D (pneumatic cell) is the only core instrument with a piezometric level that approaches the level used in the stability analysis (5450 feet m.s.l.). This instrument has produced erratic data over the years. A recent investigation revealed that there is gas flow in one direction; however, there is no flow in the other direction. This makes the data questionable.

There are no piezometers located in either the upstream fill or downstream fill to evaluate; however, Piezometer 563 located adjacent to the fill in the abutment material (downstream) has been historically dry. Additional piezometers installed in the downstream fill are warranted if a pool raise were to be implemented.

8.2 Foundation Seepage.

8.2.1 General. The overburden materials, in the valley and abutments, are of such high permeability as to require a positive form of underseepage control so that excessive losses of stored water and/or seepage concerns do not occur. The final adopted and approved form of underseepage was a backfilled trench located at the centerline of the embankment alignment. Numerous piezometers/pressure cells are

located in the upstream overburden. These instruments reflect a direct influence of the reservoir which would be as expected. All piezometric levels are within normal trend and range.

Studies of types and associated costs of underseepage controls were reported in Design Memorandum No. PC-9, Initial Earthwork -Valley Cutoff. The final adopted and approved form of underseepage control was a backfilled impervious trench located at the centerline of the embankment alignment. The limits of the cutoff trench are from station 40+00 in the right abutment to its interception with the spillway structure in the left abutment at approximate station 123+00. The bottom width of the trench was designed to be a minimum of 35 feet in the valley and 25 feet in the abutments.

The cutoff trench was intended to extend a minimum of 3 feet into the Dawson bedrock formation. After the cutoff trench was excavated to the required depth, auger holes were then drilled at a minimum of 100 feet on centers to a depth of 30 feet to explore possible continuous layers of sandstone beneath the trench. Where layers were found, they were then excavated and replaced with compacted impervious material. The approximate maximum depth considered practical to excavate was 10 feet. Due to the deeper excavations below the regular bottom of the trench, it was necessary to reduce the width of the trench to a width slightly less than the original planned width; however, in no instance was the width allowed to be less than 25 feet. During drilling when sand layers were encountered, laboratory testing was used to assist in the decision to remove or leave the sand layer. If the sand had less than 20 percent passing the number 200 sieve size and a plasticity index of less than 5, it was removed and replaced with impervious material.

Pump tests were performed on selected piezometers to further investigate the permeability and susceptibility of the Dawson Formation to seepage. Bailing and recharge measurements were made on five (5) piezometers which had been installed in the Dawson Formation. Permeabilities at each piezometer were computed from the recharge tests by the Jacobs Modification to the Theis Recovery Method of Analysis. The thickness of the aquifer was taken as the depth of water in each piezometer which in most cases was 50 feet. The derived permeabilities were considered as semi-impervious or very low. From these tests and the pressure tests, it was concluded that the Dawson Formation had a relatively low permeability and would be relatively free of under-seepage problems.

Investigations were made to determine whether seepage could be a problem either under or around the cutoff trench. It was found that any seepage that might occur under the cutoff in the sandstones or around the end of the cutoff in the right abutment would be of such a small quantity that it would not be particularly noticeable. As an added precaution to intercept seepage and/or prevent piping of impervious material, a pervious section was placed on the downstream side of the cutoff trench which was then tied to the blanket drain.

8.2.2. Instrumentation Review. A review of the bedrock piezometer data, as presented in Appendix C, revealed considerable headloss across the cutoff trench. Bedrock piezometers located at or upstream of the cut-off trench are responsive to pool fluctuations with minimal headloss while those downstream do not exhibit a definite relationship with the reservoir and are in many cases exhibiting a downward trend, with up to 60 feet of headloss (Piezometer No. 560 @ Station 68+90). All of the downstream bedrock piezometers exhibit water/pressure levels within or below the elevation of the downstream blanket drain. All piezometric levels are within (historic) normal trend and range. The following bedrock piezometers are discussed in more detail.

Cross sections of Chatfield Dam exhibiting bedrock piezometric gradients are presented on Plate Nos. C-11 thru C-14 in Appendix C.

- **Piezometer No. 536.** Piezometer No, 536 is a hydrostatic pressure cell located approximately 200 feet upstream at Station 68+85. The instrument appears to be slightly responsive to pool fluctuations; however, there has been an overall downward tend over the past 10 years indicating possible siltation upstream and/or decreased permeability of the bedrock. See Plate No. C-21, Appendix C for a detailed plot.

- **Piezometer No. 41.** Piezometer No. 41 is an open tube located approximately 25 feet upstream from the dam centerline at Station 68+90. The tip is located in the bedrock slightly upstream of the cutoff trench. This instrument is highly responsive to pool fluctuations with only 1-2 feet of headloss. See Plate No. C-22, Appendix C for a detailed plot.

- **Piezometer No. 504A.** Piezometer No. 504A is a hydrostatic pressure cell located approximately 195 feet downstream at Station 68+90. There has been an overall downward tend over the past 10 years indicating possible siltation upstream and/or decreased permeability of the bedrock. The headloss between Piezometer No. 536 located in the foundation upstream of the cut off and Piezometer No. 504A located in the foundation downstream of the cut off is approximately 10 feet. An apparent or definitive relationship with pool fluctuations has been identified as discussed in Paragraph 7.5. See Plate No. C-23 & C-23A, Appendix C for detailed plots.

- **Piezometer 505A.** Piezometer No. 505A is a hydrostatic pressure cell located approximately 400 feet downstream at Station 68+90. This instrument appears to be slightly responsive (less than 504A). There has been no trend over the past 10 years. The headloss between Piezometer No. 536 and Piezometer 505A is approximately 15 feet. No apparent or definitive relationship with pool fluctuations has been identified. See Plate No. C-24, Appendix C for a detailed plot.

- **Piezometer 505B.** Piezometer No. 505B is a hydrostatic pressure cell located approximately 400 feet downstream at Station 68+90. This instrument appears to be slightly responsive (less than 504A). The headloss between Piezometer No. 536 and Piezometer No. 505B is approximately 10 feet. No apparent or definitive relationship with pool fluctuations has been identified except for as discussed in Paragraph 7.5. See Plate No. C-25 & C-25A, Appendix C for detailed plots.

- **Piezometer 560.** This instrument is located approximately 950 feet downstream in the Dawson Formation at Station 68+90. There is approximately 60 feet of headloss as compared to the pool elevation. There has been a very slight overall downward trend over the past 10 years indicating possible siltation upstream and/or decreased permeability of the bedrock. No apparent or definitive relationship with pool fluctuations has been identified. See Plate No. C-26, Appendix C for a detailed plot.

- **Piezometer 79+00.** This instrument is an open tube device located approximately 25 feet upstream from the dam centerline at Station 79+00. The tip is located in the bedrock slightly upstream of the cutoff trench. This instrument is highly responsive to pool fluctuations with only 1-2 feet of headloss. See Plate No. C-27 for a detailed plot.

- **Piezometer 551D & 551S.** These instruments are closed tube piezometer systems equipped with a pressure gages. Prior to 2005, Piezometer 551D was experiencing a definite rise in piezometric level. In May of 2005 both gages were replaced. Since that time Piezometer 551D has read approximately elevation 5400 feet m.s.l., while 551S has read approximately 5390 feet m.s.l.

- **Piezometer No. 561.** Piezometer No. 561 is an open tube device located approximately 300 feet downstream in the bedrock at Station 93+00. The instrument appears to be responsive. There has been an overall downward tend (approximately 5 feet) over the past 10 years indicating possible siltation upstream and/or decreased permeability of the bedrock. An apparent or definitive relationship with pool fluctuations has been identified as discussed in Paragraph 7.5.See Plate No. C-28 & C-28A, Appendix C for detailed plots.

- **Piezometer 102+00/25US.** This instrument is located in the bedrock, 25 feet upstream at Station 102+54. The tip is located slightly upstream of the cutoff trench. The tip is located in the bedrock slightly upstream of the cutoff trench. This instrument is highly responsive to pool fluctuations with approximately 6 feet of headloss. See Plate No. C-29, Appendix C for a detailed plot.

- **Piezometer 486.** This instrument is an open tube piezometer located approximately 25 feet upstream from the dam centerline at Station 102+54. The tip is located in the bedrock slightly upstream of the cutoff trench. This instrument is highly responsive to pool fluctuations with approximately 4 feet of headloss. See Plate No. C-30, Appendix C for a detailed plot.

- **Piezometer 520A.** This instrument is a hydrostatic pressure cell located approximately 125 feet downstream in the Dawson Formation at Station 102+54. The instrument appears to be responsive but with no definite relationship with pool fluctuations. There has been an overall downward tend (approximately 3 feet) over the past 10 years indicating possible siltation upstream, decreased permeability of the bedrock and/or dissipation of pore pressure. The average headloss is approximately 45 feet as compared to the reservoir. No apparent or definitive relationship with pool fluctuations has been identified. See Plate No. C-37, Appendix C for a detailed plot.

Based on a review of the foundation piezometeric data, seepage within the Dawson Formation would not pose a concern under the proposed reallocation.

8.3 Overburden. The downstream overburden piezometeric data, as presented in Appendix C, revealed considerable headloss. All piezometric levels are all within historic (normal) trend and range. There has been a general slight decreasing trend observed in many of the instruments. All of the downstream overburden piezometers exhibit water/pressure level within or below the elevation of the blanket drain.

A "wet area" has been identified just upstream of the outlet works stilling basin area at various times since construction of the dam. Normally the area is dry but on rare occasions water is observed emerging the slope. Historic information points towards precipitation as the source of the wet area. Additional piezometers would be beneficial in determining the exact source(s) of the water in this area.

The following downstream overburden piezometers are discussed in more detail.

- **Piezometer 522.** This instrument is located in the overburden approximately 550 feet downstream at Station 102+54. There is no definitive relation with pool fluctuations. The average headloss is approximately 50 feet as compared to the reservoir. See Plate No. C-32, Appendix C for a detailed plot.

- **Piezometer 558.** This instrument is located in the overburden approximately 725 feet downstream at Station 102+54, which is downstream of the toe drain. There is no definitive relationship with pool fluctuations. The average headloss is approximately 45 feet as compared to the reservoir. See Plate No. C-33, Appendix C for a detailed plot.

- **Piezometer 555.** This instrument is located in the overburden approximately 625 feet downstream at Station 85+50, which is downstream of the toe drain. There is no definitive relationship with pool fluctuations. There is; however, a very slight downward trend in the piezometric level. The average headloss is approximately 60 feet as compared to the reservoir. See Plate No. C-34, Appendix C for a detailed plot.

- **Piezometer 562.** Piezometer 562 is located 443 feet downstream at Station 102+92. This instrument was abandoned in 1980.

- **Piezometer 563.** Piezometer 563 is located approximately 350 feet downstream at Station 90+00. This instrument has been historically dry. See Plate No.C-35, Appendix C, for a detailed plot.

A review of the toe drain data revealed that the drain has always been dry indicating evidence of a positive cutoff; however, higher reservoir elevations have not been experienced to assess the overall effectiveness of the cutoff.

Based on a review of the overburden piezometeric data, seepage within the overburden would not pose a concern under the proposed reallocation.

9.0 Movement Review

9.1 General. A settlement analysis was performed (during design) on the embankment impervious core and impervious cutoff trench at station 95+00 where the maximum height of impervious core and cutoff trench was attained. Since the surface foundation clays beneath the embankment were removed, it was determined that settlement of the pervious foundation would take place during construction and that the critical materials for determining residual settlement or settlement after completion of construction would be the embankment core and cutoff trench materials.

Consolidation tests of the impervious cutoff trench material were used in the settlement analyses. The total height of material analyzed was 187 feet. The total settlement was computed to be 5.6 feet of which 3.2 feet (56%) would occur during construction and 2.4 feet (44%) would occur after completion of the embankment. Only 9 inches of overbuild were actually provided for residual settlement of the embankment. This was an arbitrary decision which recognized that a gravel road on top of the dam would provide additional buildup of the crest and that there usually is some conservancy in the computations and procedures of analyses, due to inaccuracies of testing and differences in rate of load application to that assumed.

9.2 Instrumentation Analysis. Typical movement data (survey) is presented in Appendix D for information. Plate Nos. D-1 through D-4 presents vertical movement of crest, slope and toe movement markers. Vertical movement plots of the crest movement markers indicate a maximum consolidation of the embankment and cutoff trench of approximately 0.75 feet (Point C-15) from November 1975 to 2007.

Horizontal movement data for the crest, slope and toe markers is presented on Plate Nos. D-5 through D-9. Up to approximately 0.2 feet of horizontal movement has been observed with the exception of Points S-1 and S-2 as of the 2007 survey. Point S-1 has experienced approximately 0.30 feet of movement in the south direction while Point S-2 has experienced approximately 0.7 feet of horizontal movement also in the south direction. These points are located on the downstream (north) side of the

overbuild section. Movement of these points to the south would not be consistent with their locations. Point S-1 has exhibited erratic data from the early 1980's. Possibly these points have been disturbed/damaged.

Foundation settlement data at Stations 70+00 and 90+00 is presented on Plate Nos. D-10 and D-11. Foundation settlement from 1971 to 2007 was measured at approximately 2.5 feet with the majority (1.5 feet) occurring between 1971 and 1975. Currently, the crest markers are exhibiting a movement rate of approximately 0.01 feet per year while the settlement gage movement was essentially zero from 2001 to 2003. Settlement since 1980 has been approximately 0.5 feet or 0.02 feet per year. Current (2007) top of dam centerline surveys indicates a minimum elevation of 5526.729 at Station 65+00 as shown on Plate No. D-12. The design top of dam elevation is elevation 5527 feet m.s.l. This results in a low area approximately 3.25" below the design elevation.

Survey movement data of the outlet works including the intake structure bridge is presented on Plate Nos. D-13 through D-16. As can be observed, no excessive or out of trend data has been identified.

There are 12 slope inclinometers (tiltmeters) located at Chatfield Dam as shown on Plate No. D-17. An inclinometer is a metal tube approximately 3 inches in diameter that is placed in a drill hole through the embankment/abutment/foundation. The purpose of this instrument is to measure active subsurface horizontal movement at various depths and identify areas of active instability or potential instability. Inclinometer data is presented for information in Appendix D, Plate Nos. D-18 thru D-31. A review of the inclinometer data indicates no active movement zones and no apparent movement related to pool thrust. Inclinometer Inclinometer 496 located at Station 104+54, Range167 D/S (Plate D-23) indicates what could be perceived as "movement zone" in the east direction. A review of the cumulative deviation plot (shape of pipe) on Plate D-23A revealed a sharp bend at this location resulting in poor quality data. This bend probably occurred during installation and/or settlement of the pipe. Inclinometer 545 located at Station 81+10, Range 17 D/S (Plate D-29) displays somewhat erratic data. The data does not appear to be related to movement of the embankment/foundation. The cumulative deviation (shape of pipe) plot for Inclinometer 545, presented on Plate No. D-30, indicates considerable deviation (approximately 6.5 feet) of the pipe from vertical. This normally affects the quality of the readings because just a slight change in depth produces a big change in the slope of the pipe. The checksums presented on Plate D-31 indicates relatively poor data below the depth of approximately 140 feet. This corresponds to the substantial change in the deviation of the pipe as shown on Plate D-30.

Based on a review of survey and inclinometer data, movement (settlement/instability) would not pose a concern under the proposed re-allocation.

10.0 Structural Evaluation.

10.1 General. The purpose of the structural evaluation was to determine if it is feasible to raise the normal and maximum pool elevations at Chatfield Dam without requiring modification to the existing outlet works structure and/or spillway structure. Pool elevations used in the original design calculations were compared with the proposed new pool elevations and the outlet works structure was evaluated to project how these new pool elevations may affect the existing structures. A brief description of the structures located at Chatfield Dam is presented in Paragraph 3.0.

10.2 Critical Structures. Critical structures are defined as those who's failure during or immediately after an earthquake could result in loss of life. The ability to lower the reservoir pool following an earthquake may be required to relieve pressure head on a damaged embankment or to inspect and repair the embankment in order to prevent loss of pool.

The existing guidance available on whether the intake structure should be classified as critical or not states that the intake structure is considered critical if any of the following four scenarios is likely to occur. (I) The intake structure can no longer discharge water and the embankment is damaged, but not breached right away leading to long-term use of the spillway. This could cause severe erosion and failure of the spillway, the abutment, or the embankment dam. (II) The embankment does not fail right away but is damaged such that the hydraulic capacity of the spillway is reduced because of the need to restrict the pool level to prevent overtopping. In this scenario, the outlet works must be used to prevent overtopping the dam for the design flood event. (III) The outlet works must be used to draw down the pool to a level below the spillway crest to prevent piping/internal erosion in the embankment. This assumes that the intake structure is capable of draw down rates in the range of 3 to 5 feet per day. (IV) Failure of the intake structure could cause the outlet conduit to rupture leading to piping and eventual failure of the embankment. Determination of a critical/non-critical rating will be pursued as part of the Seismic Phase I Study.

10.3 Study Data.

10.3.1 Intake Structure. The intake structure is the primary outlet structure at Chatfield Dam. In the event of a large flood event, it is imperative that the intake structure sustains functionality; however, this may not be the case depending on the results of the Phase I Seismic Evaluation. In the original design, the stability of the intake structure was designed for 4 different cases as shown below:

Case I. Construction Condition.

Reservoir Empty
Dead Load of Structure (Including Bridge Reaction)
Earth Loads
Wind Load (In Direction to produce most severe foundation pressures)

Case II. Normal Operating Condition.

Reservoir at elevation 5426 (Conservation Pool)
All Gates Open
Dead Load of Structure
Earth Loads
Wind Loads
Full Uplift on Base

Case III. Full Flood Condition.

Reservoir at Elevation 5500 (Crest of Ungated Spillway)
Emergency Gates Closed
Dead Load of Structure
Earth Loads
Wind Load
Full Uplift on Base

Case IV. Maximum Flood Condition.

Reservoir at Elevation 5521.6 (Max Reservoir Elevation)
Emergency Gates Closed
Dead Load of Structure
Earth Loads
Wind Load
Full Uplift on base

Note: In addition to these load conditions. Case IA, IIA, and IIIA, as presented on Plate E-1, were analyzed in the same way, except with an earthquake load of 0.10g substituted for the wind load.

The original stability calculations for Cases I through IV as well as IA through IIIA can be seen on attached Plate No. E-1 located in Appendix E. For the purposes of this study, Case I, the construction condition, does not apply and does not need to be looked at for the new pool volumes. Also, Case III, the Full Flood Condition, remains the same as it was in the original design of the intake structure. Case I originally yielded a shear-friction sliding safety factor of 7.56 (or 6.36 for earthquake), and Case III yielded a safety factor of 13.06 (or 6.33 for Seismic). Both cases yielded high safety factors which provide assurance that the structure would perform well in a large flood event.

Cases II and IV are the most important calculations to investigate to check if raising the normal pool elevation at Chatfield Dam is a feasible option. Case II designates a reservoir pool elevation at 5426.0 feet m.s.l. In the original design, Case II calculations yielded a shear-friction sliding safety factor of 13.64 under normal operating conditions and a safety factor of 7.73 when seismic loading was considered. The structural stability of the intake structure is marginally more stable

under the Case II loading conditions with the new pool elevation of 5444.0 feet m.s.l. The increased stability of the intake structure can be attributed to the increase in vertical water loads on the intake structure when raising the pool versus slightly increased horizontal loads improving the stability of the structure. This case will be looked at as a part of the Phase I Seismic Evaluation.

Case IV specifies the reservoir elevation at max elevation of 5521.6 ft. In the original design calculations, Case IV yielded a sliding stability safety factor of 13.0. Case IV causes no major concerns for the Re-Allocation; however Case IV will be reviewed in the seismic review since seismic forces were not originally considered in this case.

Due to the possibility of high hydrostatic head, the intake structure was originally designed with walls and components with adequate internal strength to satisfy the allowable shear and moment requirements. All structures were designed with enough steel reinforcement to meet minimum temperature and shrinkage requirements. Also, the working stresses for Cases I and IV and all cases with earthquake loads were increased by 1/3rd for conservatism. In the most recent Periodic Inspection Report (2008), it was noted that the intake structure's concrete surfaces were in excellent condition with only minor shrinkage and map cracking which ensures that the structure should function as designed.

The bulkheads, gates, and valves in the intake structure were all originally designed to handle max flood conditions as well as the max pool of 5521.6 feet MSL. In fact, the design of the intake structure was based on the loading cases used for stability or a combination of loadings which produced the most severe stresses. These structures are expected to perform just as well as they currently do in a max pool event.

The oblong conduit was originally chosen over circular conduits because it was much more economical to build and was comparable to the circular conduit in strength. This conduit also was designed for the most severe loading conditions and will not be affected by the change in pool elevations entailed in this study.

The stilling basin was originally designed for a worst case scenario flood discharge condition of 8150 c.f.s. which is equivalent to the max discharge possible to pass through the outlet works. This structure also should not be of any concern with the newly proposed pool elevations.

The intake structure service bridge was designed for the maximum pool loading condition. In addition to these loading conditions the structure was designed to bear the AASHTO H-20 live load or a 25-ton mobile truck crane (37.5 tons total load), plus a low boy parked simultaneously on the same span as the crane. The service bridge is not a concern under the original loading criteria with a new normal pool elevation of 5444 Ft. MSL.

10.3.2 Spillway Structure. The spillway structure at Chatfield Dam was originally designed for similar loading conditions as the outlet works structures. The original maximum flood that the spillway was designed for is the only condition that would change due to raising the pool elevation according to this study. However, similar to the outlet works structures, there are no major concerns regarding the structural integrity of the spillway structures.

11.0 Conclusions. No immediate dam safety concerns have been identified based on either the existing normal reservoir elevation of 5432 feet m.s.l. or on a projected reservoir elevation of 5444 feet m.s.l considering static loading.

No indications of instability have been identified either by field inspections or by the instrumentation program after approximately 35 years of service.

Based on fluctuation of piezometer data as discussed hereinbefore, there is the potential concern for development of excess pressures in the foundation (Dawson Formation) at the reservoir elevation of 5500 feet m.s.l (spillway crest pool). Based on piezometer projections, the piezometric levels in both Piezometer 504A and 561 may exceed the levels used in the original slope stability analyses for pool elevation 5500 feet m.s.l. This does not pose a concern for the re-allocation project since the spillway pool elevation does not change as a result of the re-allocation project. This concern should be evaluated as a part of the routine dam safety program.

Due to the relatively responsive nature of some of the piezometers located in the core of the embankment, close monitoring of these instruments would be warranted during a reservoir level approaching the spillway crest (El. 5500 Ft.). This does not pose a concern for the re-allocation project since the spillway pool elevation does not change as a result of the re-allocation project. This concern should be evaluated as a part of the routine dam safety program.

No evidence of seepage concerns have been identified since construction of the project (35 years). All piezometric levels are considered to be within normal trend and range. The downstream toe drain has always remained dry. Minimal fluctuation except for a general overall decline in pressure has been observed in the bedrock piezometers located near the downstream toe (Piezometers 560, 557, 564, and 522). The most responsive downstream bedrock instrument is Piezometer 561. Piezometer 561 is an open tube devices located at Station 93+00, approximately 300 feet downstream. The piezometric level exhibited by this device initially reflected pore pressure during construction and then was affected by the reservoir impoundment. The piezometric level in this instrument has decreased approximately 15 feet since impoundment of the reservoir; however, small spikes in the piezometric level can be seen during periods of increased reservoir level (1979). This instrument has fluctuated approximately 2 feet over the past 10 years during “normal” fluctuations of the reservoir (approximately 10 feet). Currently the piezometric level in this instrument is at the approximate elevation of the bottom of the blanket drain.

All of the structures at the site have been designed to withstand the small increase in loading caused by the proposed pool elevations. In addition to this, the most recent periodic inspection report (2008) found these structures to be in very good condition which provides confidence that these structures are still in a condition to function as designed. A review of the instrumentation data (piezometers, inclinometers and survey points) did not reveal a relationship between movement (potential instability) and thrust of the pool; however, the maximum pool elevation to date has been approximately 5447.58 and this was for a relatively short time period.

The primary concern for the structures at Chatfield Dam stems from the most recent Seismic Safety Review which recommended a Phase I study.

Although continued monitoring will be required, raising the normal pool elevation by up to 12 feet should not have a direct bearing on the adequacy of the slope protection material; however, the slope protection material will continue to be monitored during routine dam safety inspections (monthly, annual, periodic,…). In addition to this, effort will be made to inspect existing areas of riprap displacement during low reservoir elevations.

Based on a review of design assumptions, instrumentation data and performance since completion of Chatfield Dam, it is concluded that the new "normal" pool elevation (El. 5444) proposed in this reallocation study will not adversely impact the integrity of the embankment or structures. It is emphasized that this conclusion is based strictly on static loading scenarios. Although no dam safety concerns have been identified for the proposed reservoir loading, based on project performance and the instrumentation program, increased monitoring of the project would be pursued as part of the routine dam safety program to assure continued safe operation of the dam. This would include the development and implementation of a Reservoir Raise Monitoring Plan which would include additional inspection efforts, instrumentation data acquisition and data analysis. The Project Surveillance Plan and Emergency Action Plan should also be updated as appropriate.

Installation of additional instrumentation prior to the pool raise along with an increase in instrumentation readings and inspection frequencies during and following the pool raise would be warranted.

12.0 Recommendations. Specific recommendations are as follows:

- Add new piezometers in the overburden near the outlet works at Station 105+00.
- Add new piezometers in the downstream drain at Stations 69+00, 81+00, 93+00 and 102+00 to establish elevation and depth of the drain and to monitor effectiveness of the drain.
- Add new piezometers in the downstream fill at Stations 69+00, 81+00, 93+00 and 102+00 to monitor an historic "wet area".
- Add new piezometers in the downstream Dawson Formation at Stations 69+00, 81+00, 93+00 and 102+00 to monitor potential excessive pressures.

- Develop a reservoir raise monitoring plan addressing frequencies of inspection, instrumentation data acquisition including the toe drain and data analysis.
- Revise the Emergency Action Plan and Surveillance Plan as appropriate.
- Maintain routine (monthly, Annual, Periodic) inspection of the upstream slope protection. Effort should be made to inspect existing areas of riprap displacement during low reservoir elevations.

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ER1110-2-1155, 12 Sept 1997, Dam Safety Assurance Program

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Supplement to Design Memorandum No. PC-24, Embankment and Excavation, December 1970.

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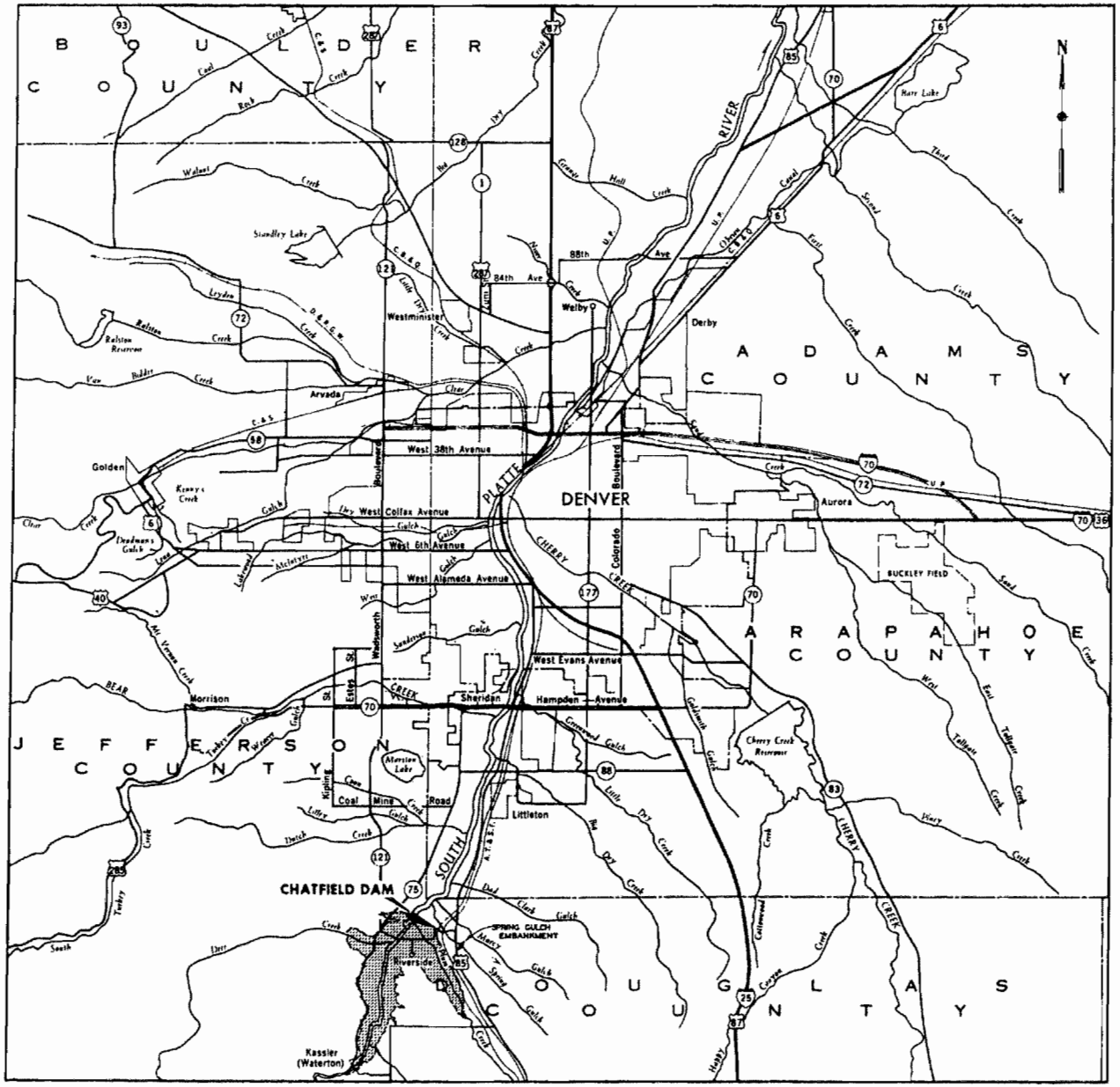
USACE, Omaha District, Construction Foundation Report, 1979

Meeting of Board of Consultants, Chatfield Dam, 16 Nov 1971

ACRONYMS

approx.:	approximate
cfs:	cubic feet per second
Cfs:	Cubic feet per second
COE:	Corps of Engineers
D/S:	Downstream
Elev. (el):	Elevation
ER:	Engineering Regulation
Ft, FT, ft:	feet
H	Horizontal
in.:	inch
lb/cft:	pounds per cubic foot
max.:	maximum
min.:	minimum
msl	mean sea level
N:	North
Nos.:	Numbers
P:	Piezometer
psf:	pounds per square foot
psi:	pounds per square inch
Pz:	Piezometer
rt	right
SC:	Slope and Crest Movement Marker
SG:	Settlement Gage
sq in.:	square inch
Sta.:	station
T:	Inclinometer designation
U/S:	Upstream
USACE:	United States Army Corps of Engineers
V:	Vertical
W:	West
wt:	weight
yrs:	years



VICINITY MAP

SCALE IN FEET

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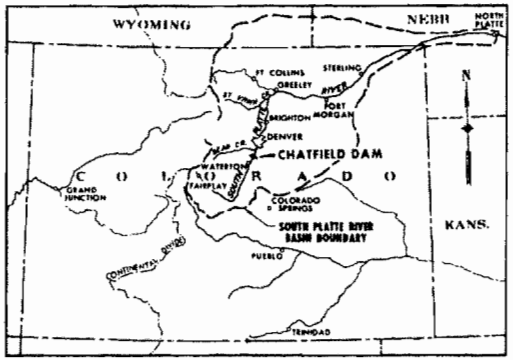


LOCATION MAP

NO SCALE

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER, COLORADO

CHATFIELD DAM AND RESERVOIR

EARTHWORK - STAGE III

VICINITY AND LOCATION MAPS

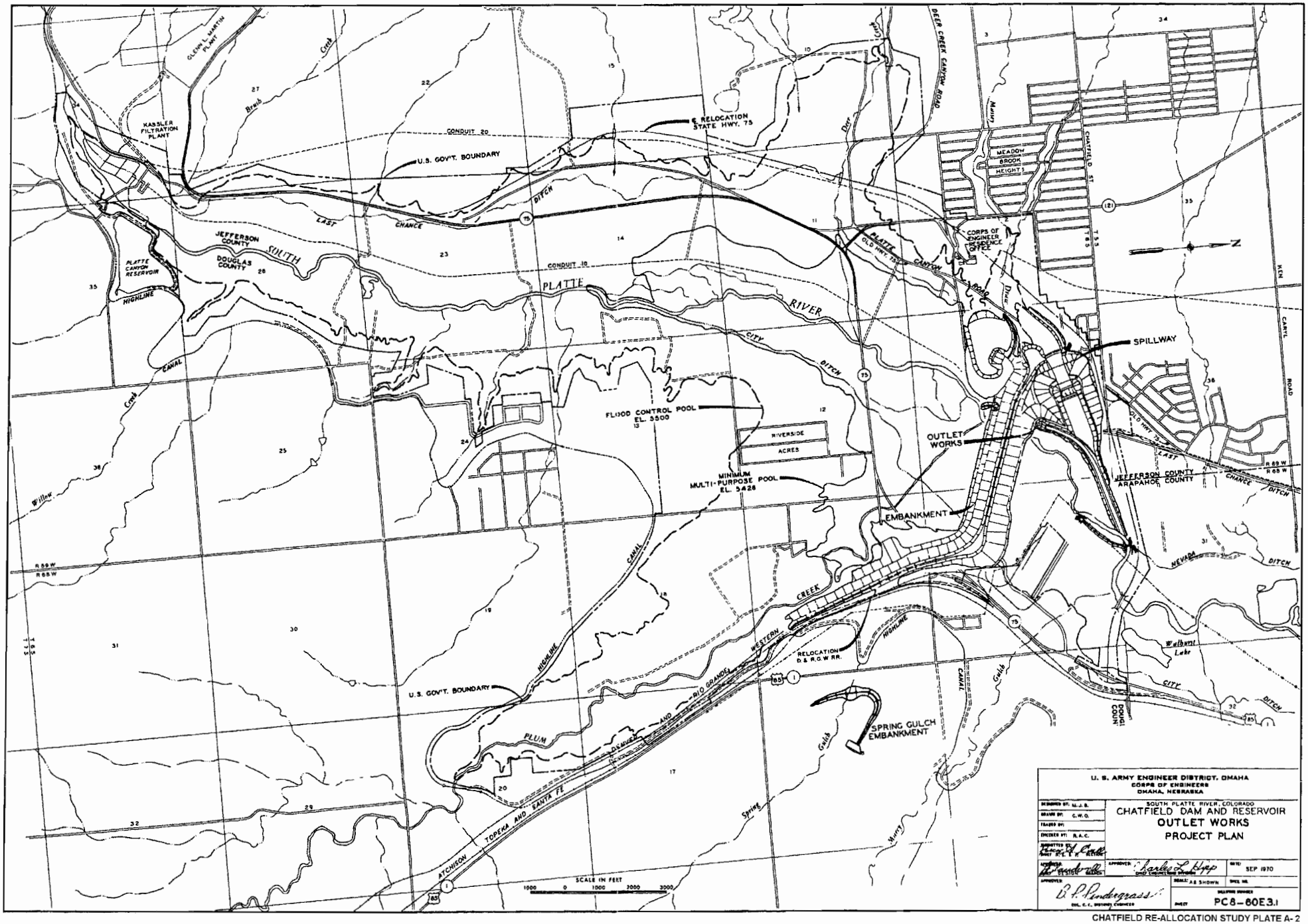
DESIGNED BY: R. J. M.
DRAWN BY:
TRACED BY:
CHECKED BY: C. V. J.

DATE: JAN. 1970

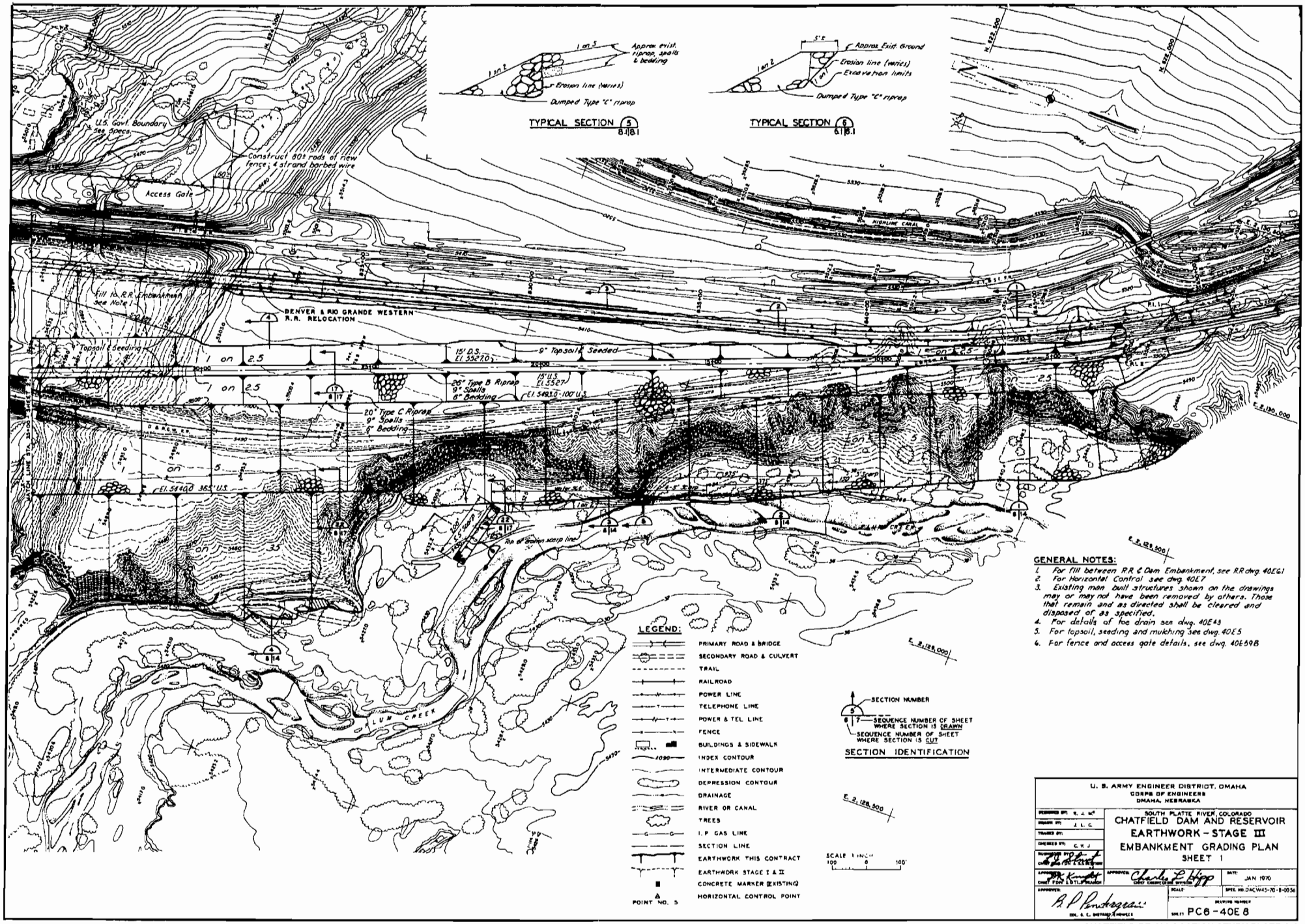
SCALE: AS SHOWN

SHEET PC6 - 40E1

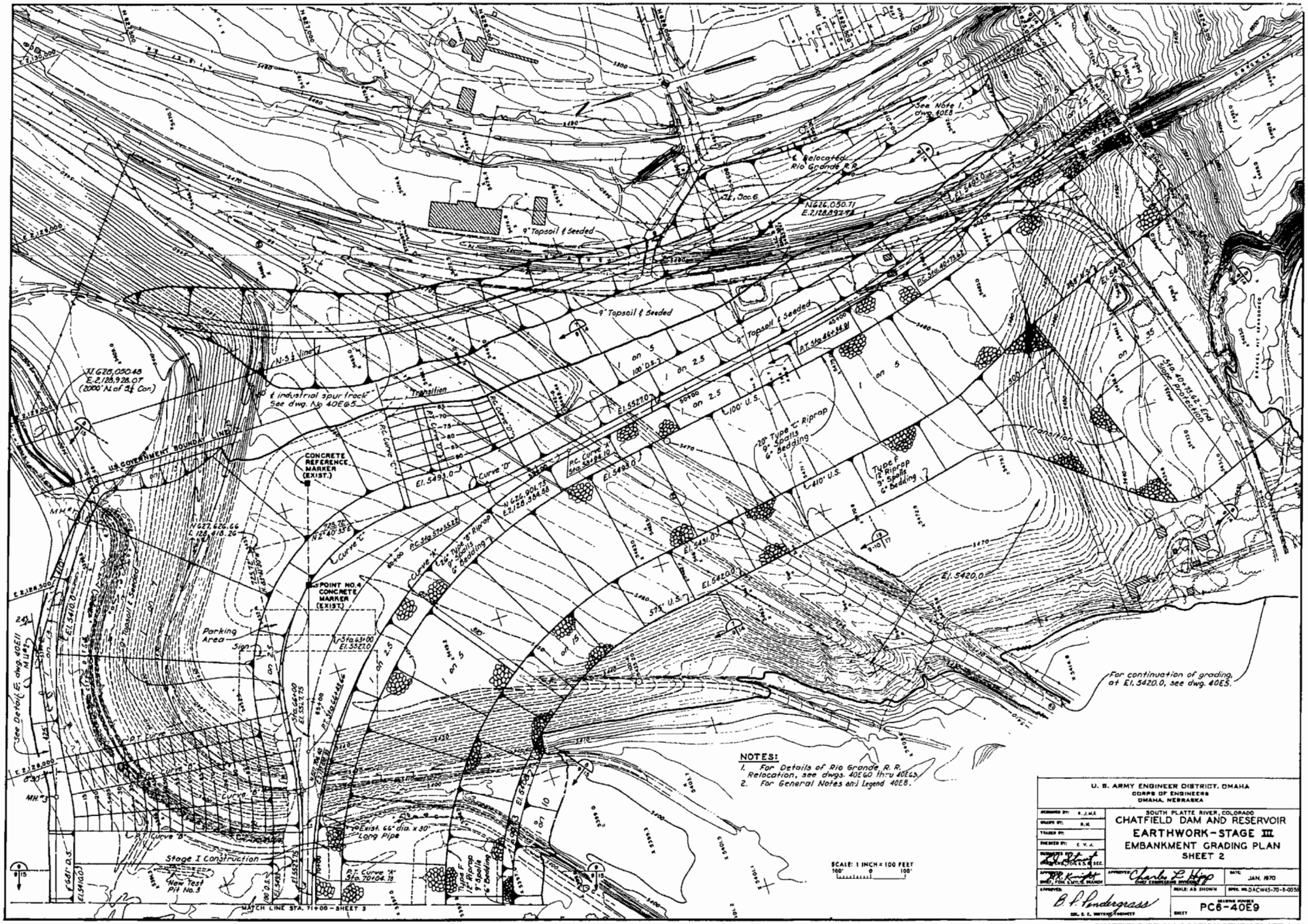
CHATFIELD RE-ALLOCATION STUDY PLATE A-1



CHATFIELD RE-ALLOCATION STUDY PLATE A-2



CHATFIELD RE-ALLOCATION STUDY PLATE A-3



NOTES:

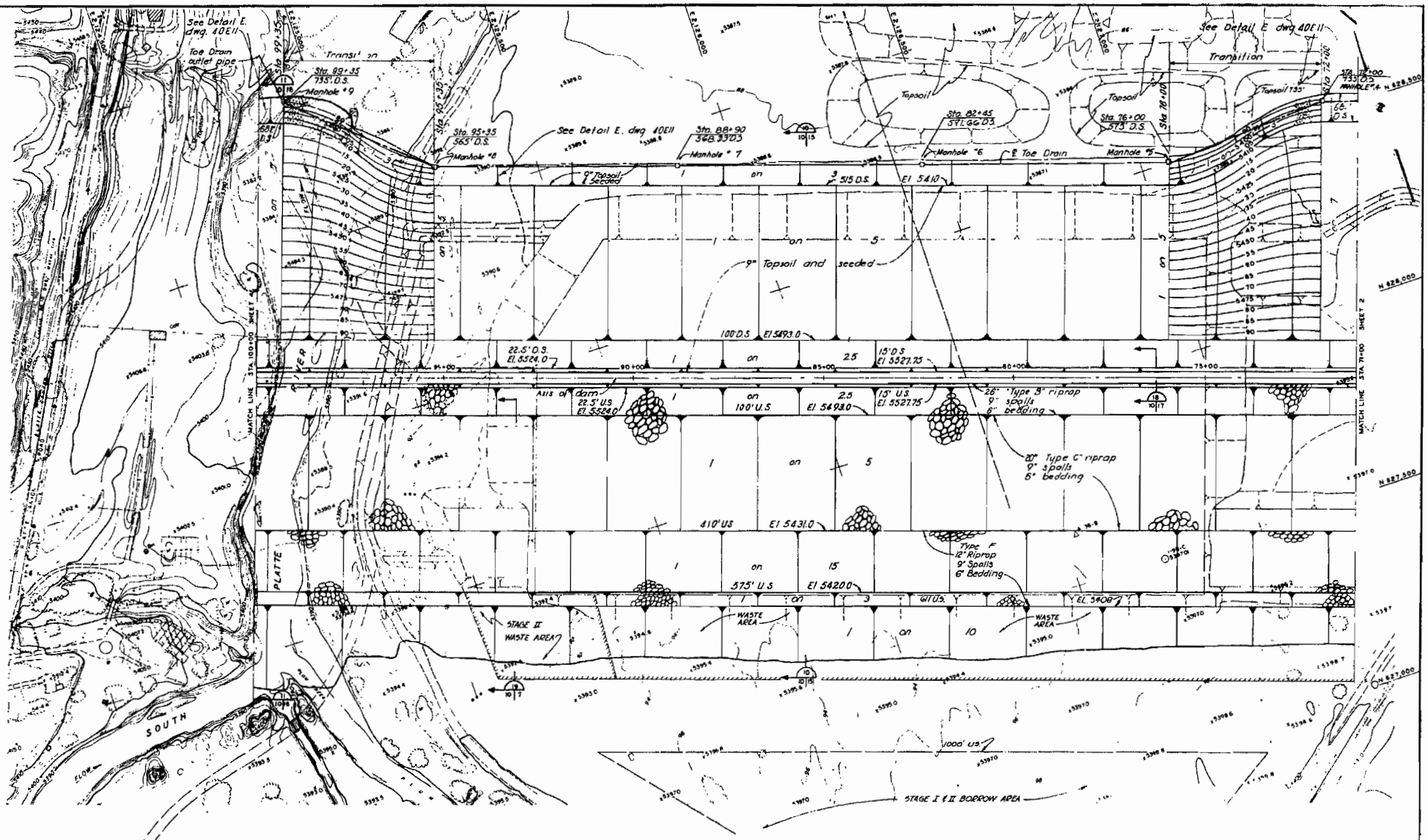
1. For Details of Rio Grande R. R. Relocation, see dwgs. 40E60 thru 40E63.
2. For General Notes and Legend 40E8.

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EARTHWORK - STAGE III
EMBANKMENT GRADING PLAN
SHEET 2

DESIGNED BY: R. J. M. E.
DRAWN BY: R. M.
TRACED BY:
CHECKED BY: E. V. J.
SUBMITTED BY: R. J. Knight, CHIEF, STRUCTURES SEC.
APPROVED: R. P. Knight, CHIEF, FOUN. & MTLS. BRANCH
APPROVED: Charles L. Hipp, CHIEF, ENGINEERING DIVISION
DATE: JAN. 1970
APPROVED: B. P. Pendergrass, COL. C. E. DISTRICT ENGINEER
SCALE: AS SHOWN
SPEC. NO. DACW45-70-B-0056
SEQUENCE NUMBER SHEET: PC6-40E9

CHATFIELD RE-ALLOCATION STUDY PLATE A-4



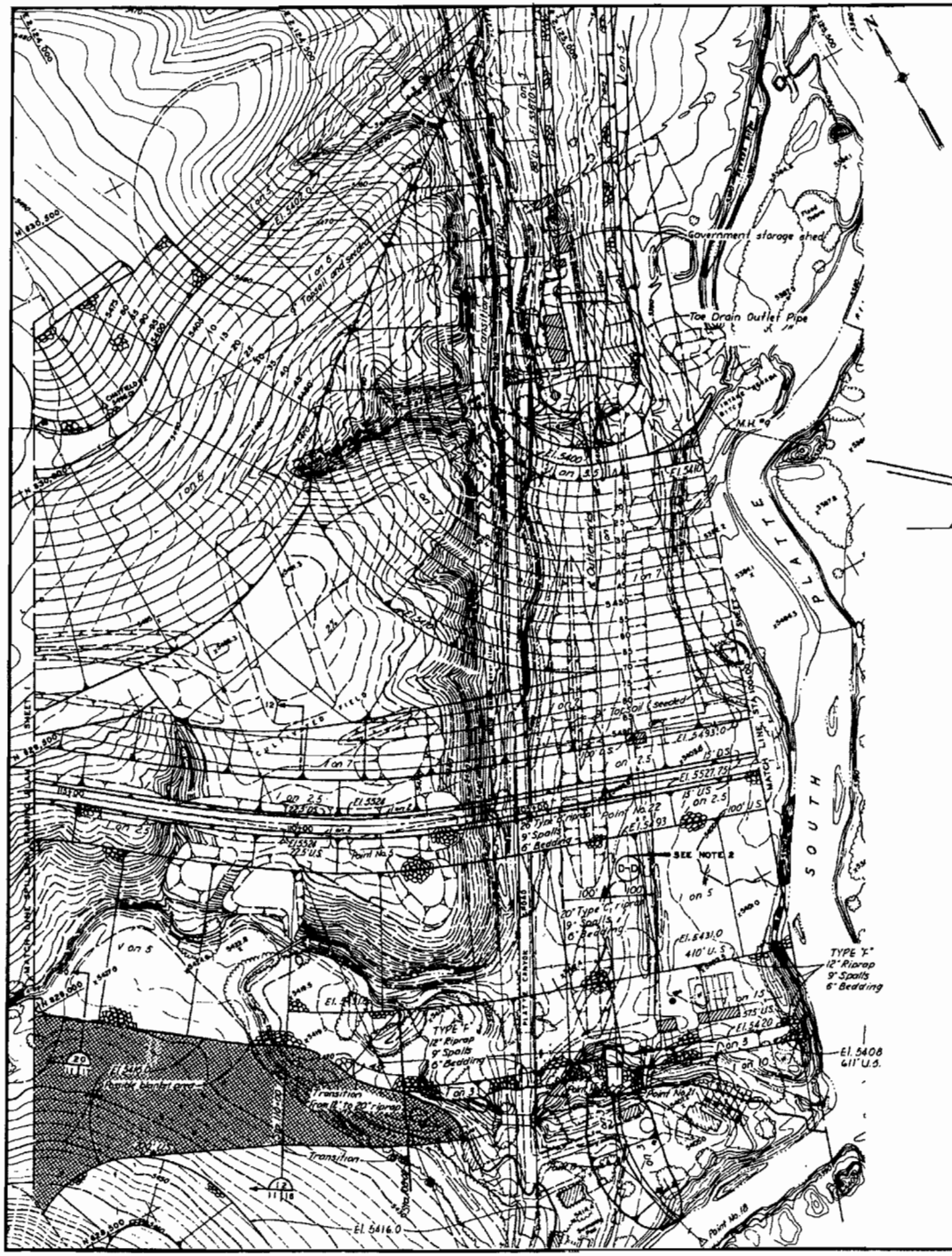
NOTE:
1 For General Notes and Legend, see dwg. 40E8.

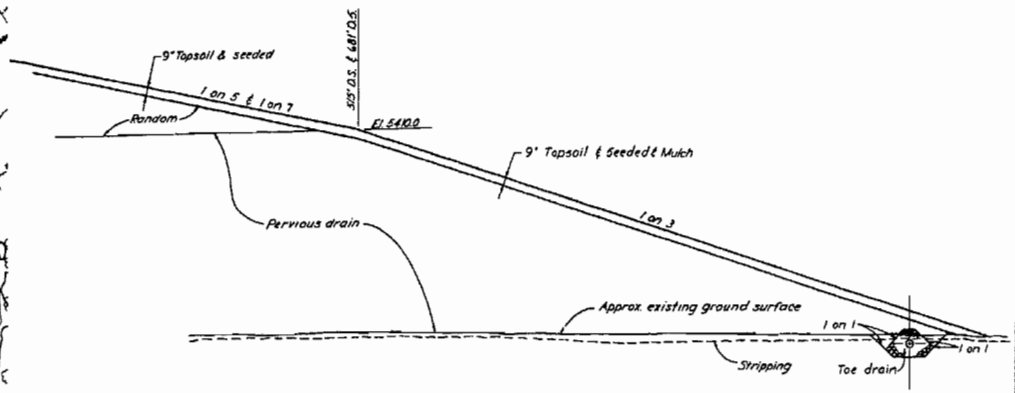
SCALE 1 INCH = 100 FEET
100 0 100

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EARTHWORK - STAGE III
EMBANKMENT GRADING PLAN
SHEET 3

DESIGNED BY R. J. M. S.
DRAWN BY R. L. M.
TRACED BY
CHECKED BY C. V. J.
SUPERVISED BY
APPROVED: CHIEF, DESIGN BRANCH
APPROVED: Charles E. Hipp CHIEF, ENGINEERING DIVISION
DATE JAN. 1970
SCALE
UNIT NO. DACW45-70-B-0054
APPROVED: C. P. Sundergrass DEPUTY CHIEF ENGINEER
DRAWING NUMBER SHEET PC6 - 40E10





DETAIL

NOTES:

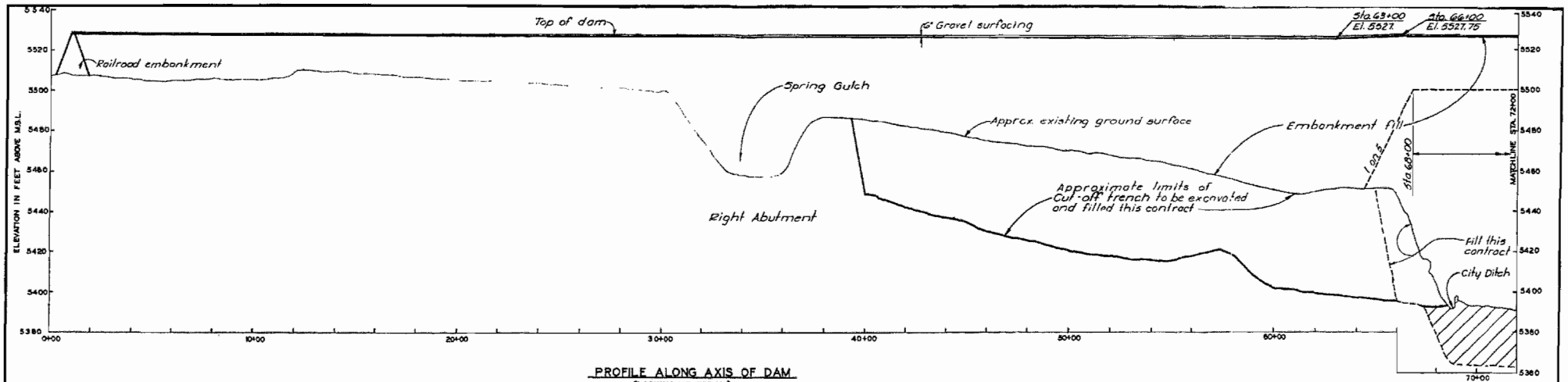
1. For General Notes and Legend, see dwg 40E8.
2. Area D-D is construction area for Service Bridge by others, see section 1 area use, for coordination of work.
3. For grading of the Downstream Outlet Works Channel, see Dwg. E 23.1 P/3.5.

SCALE 1 INCH = 100 FEET
100 0 100

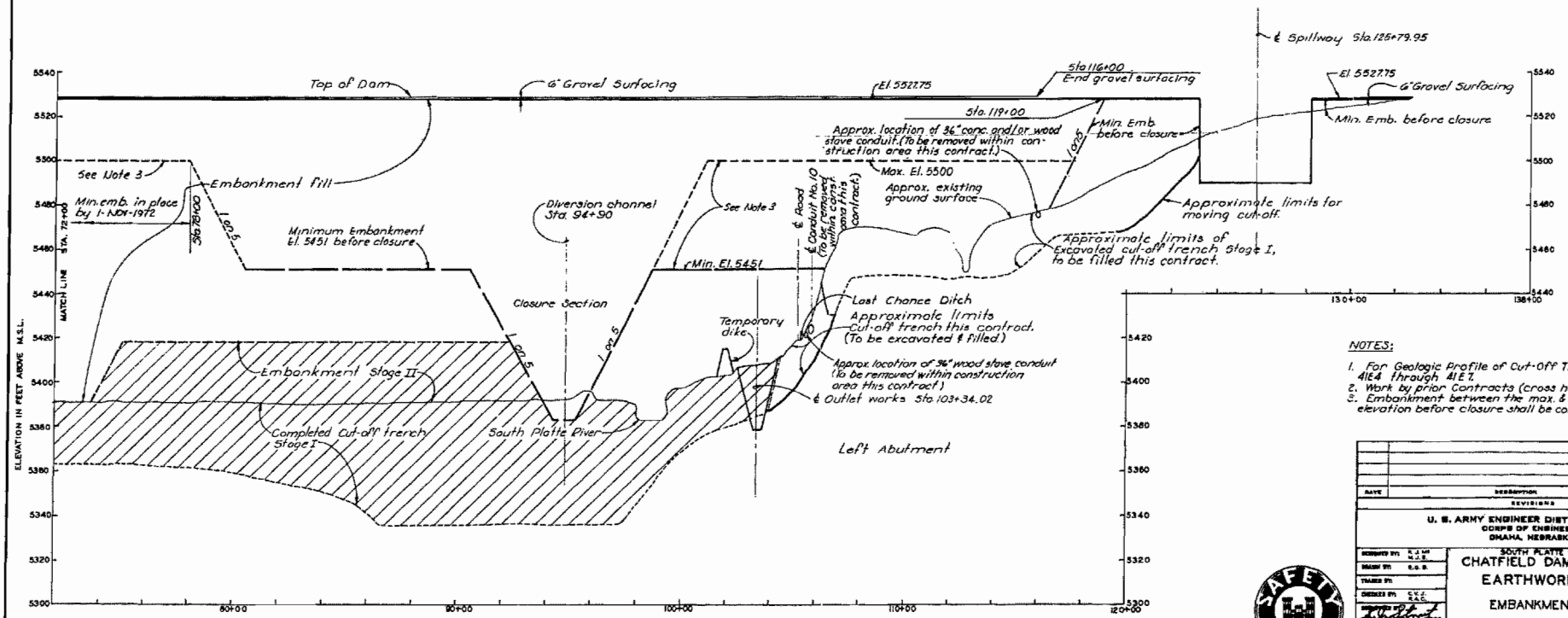
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EARTHWORK STAGE III
EMBANKMENT GRADING PLAN
SHEET 4

DESIGNED BY: R. J. M.
DRAWN BY: J. N. W.
TRACED BY:
CHECKED BY: G. V. A.
SUBMITTED BY: Chief, Design Section
APPROVED: Chief, Design Branch
APPROVED: Chief, Topographic Branch
APPROVED: Col. C. L. District Engineer
DATE: JAN. 1970
SCALE AS SHOWN
SPEC. NO. DACW45-70-B-0036
DRAWING NUMBER
SHEET PC6-40E11



PROFILE ALONG AXIS OF DAM
(LOOKING UPSTREAM)



PROFILE ALONG AXIS OF DAM
(LOOKING UPSTREAM)

SCALE: VERT. 1 INCH = 20 FEET
HORIZ. 1 INCH = 200 FEET

NOTES:

1. For Geologic Profile of Cut-Off Trench, see dwgs. 41E4 through 41E7.
2. Work by prior Contracts (cross hatched).
3. Embankment between the max. & min. embankment elevation before closure shall be completed as directed.

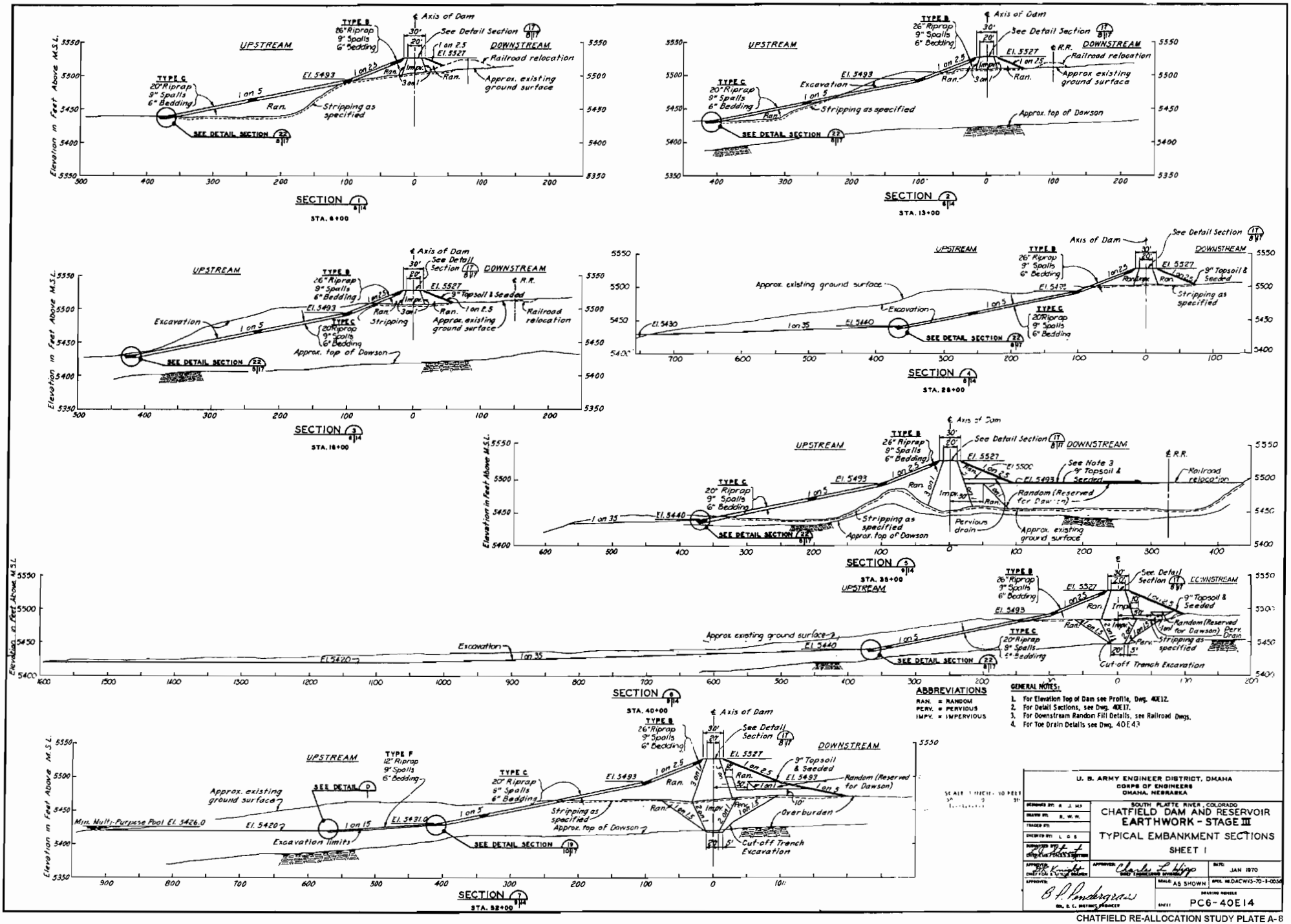
DATE	DESCRIPTION	MADE	APP'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EARTHWORK - STAGE III
EMBANKMENT & PROFILE

DESIGNED BY: R.J.M. M.J.S.
DRAWN BY: E.G.B.
TRACED BY:
CHECKED BY: C.K.Z. R.A.S.
SUBMITTED BY: Chief, Div. Design Branch
APPROVED: Chief, Div. Design Branch
APPROVED: Chief, Div. Design Branch
DATE: NOV. 1971
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.



ABBREVIATIONS

RAN. = RANDOM
PERV. = PERVIOUS
IMPV. = IMPERVIOUS

GENERAL NOTES:

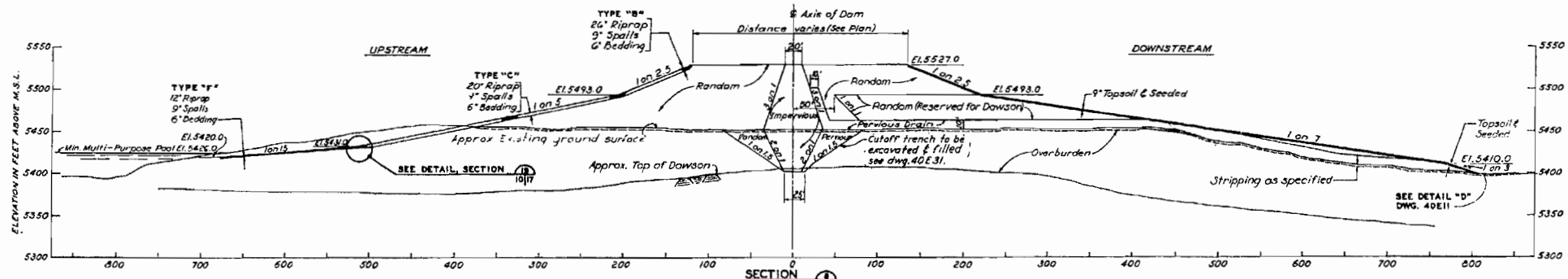
1. For Elevation Top of Dam see Profile, Dwg. 40E12.
2. For Detail Sections, see Dwg. 40E17.
3. For Downstream Random Fill Details, see Railroad Dwgs.
4. For Toe Drain Details see Dwg. 40E43

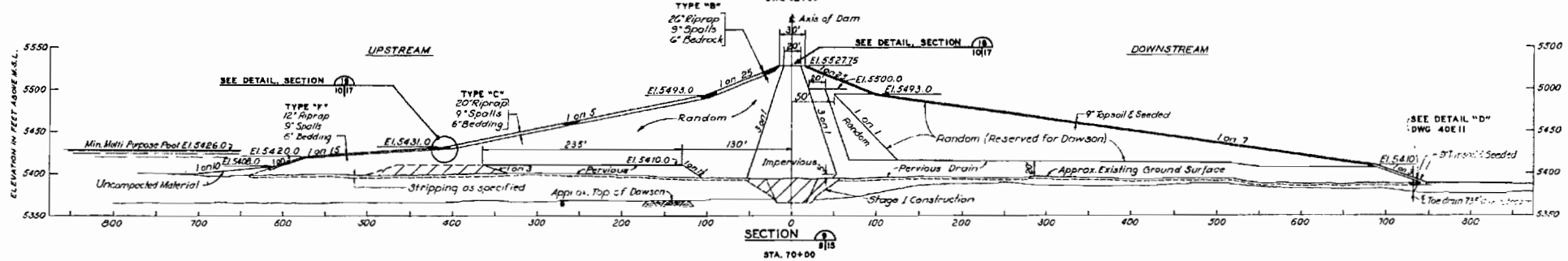
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

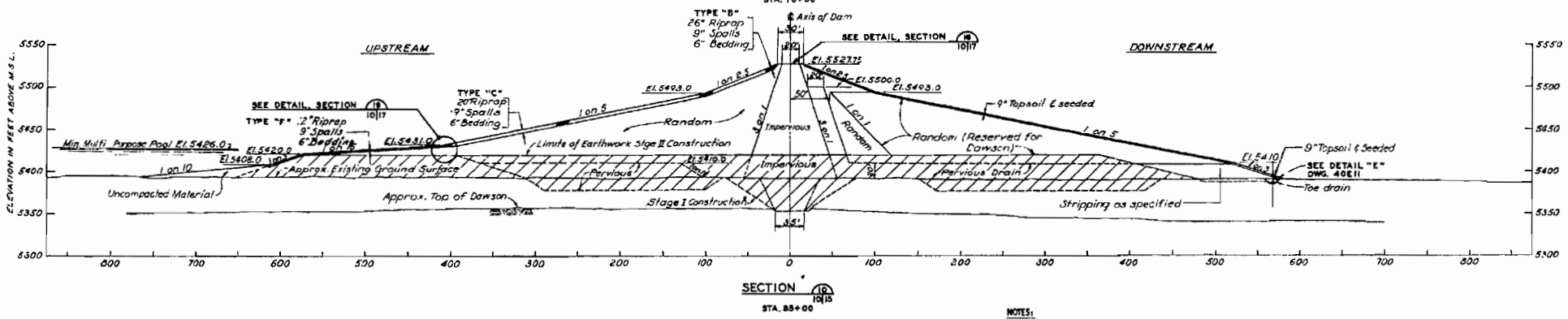
SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EARTHWORK - STAGE III
TYPICAL EMBANKMENT SECTIONS
SHEET 1

DESIGNED BY: R. J. M.S.
DRAWN BY: R. W. W.
TRACED BY:
CHECKED BY: L. G. S.
SUBMITTED BY: Chief, Div. Hydraulic Section
APPROVED: Chief, Chatfield & Cherry Creek Branch
APPROVED: Chief, Civil Engineering Division
APPROVED: B. P. Pendergrass, Col., U. S. Army, District Engineer
DATE: JAN 1970
SCALE: AS SHOWN
SPEC. NO. DACW45-70-B-0056
DRAWING NUMBER SHEET: PC6-40E14

CHATFIELD RE-ALLOCATION STUDY PLATE A-8







NOTES:

1. Pervious Drain to Transition from 10' Depth at Sta. 68+00 to a 20' Depth at Sta. 69+00.
2. Work by Prior Contracts (Cross Hatched).

SCALE: 1 INCH = 50 FEET
50' 0 50'

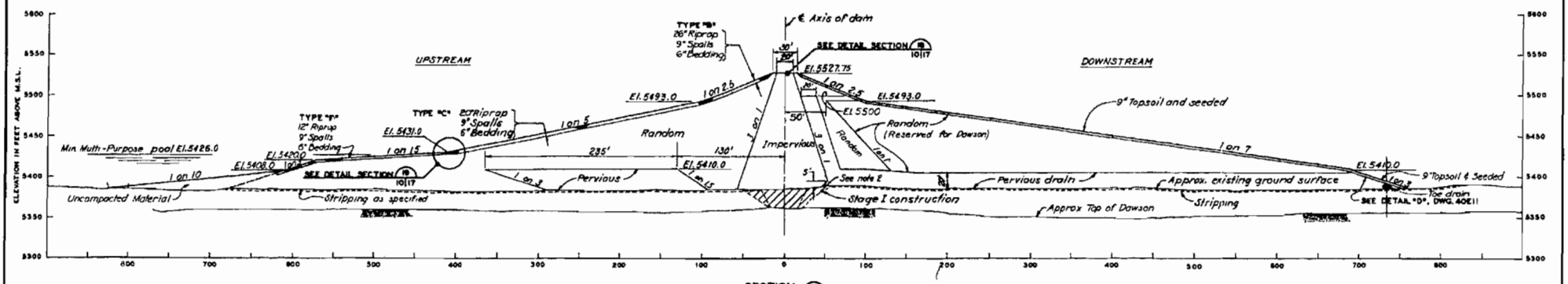
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: R. J. M.S.
DRAWN BY: A. S.
TRACED BY:
CHECKED BY: L. G. S.
SUBMITTED BY: Stephen Knight
APPROVED: W. Knight
CHIEF, DAM & WTRWAY DESIGN
APPROVED: R. P. Pendergrass
CHIEF, S. E. DESIGN BRANCH

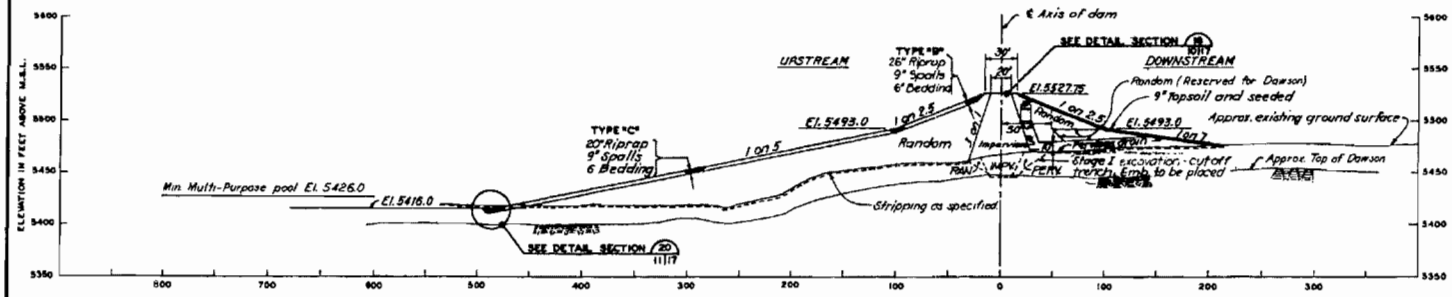
SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EARTHWORK - STAGE III
TYPICAL EMBANKMENT SECTIONS
SHEET 2

APPROVED: Charles L. Hipp
CHIEF, ENGINEERING DIVISION

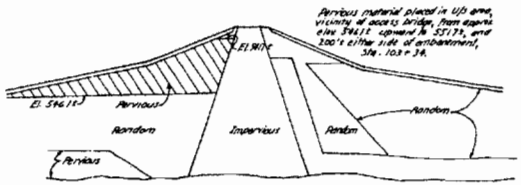
DATE: JAN. 1970
SCALE:
SPEC. NO. DACW45-70-B-0036
DRAWING NUMBER
SHEET PC6 - 40E15



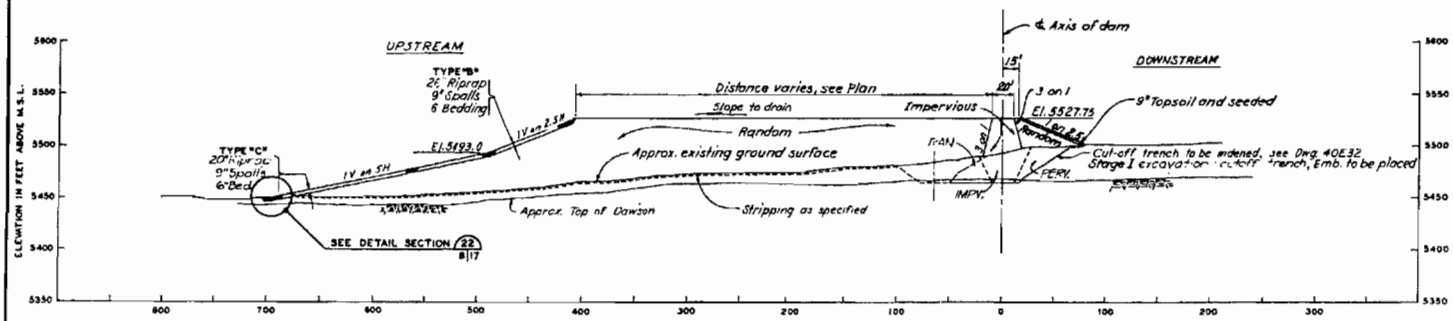
SECTION 11 10|16
STA. 99+35



SECTION 12 11|16
STA. 110+00



STA. 103+34.02
No Scale



SECTION 13 22|16
STA. 121+00

NOTES:

1. Pervious drain to transition from 20' depth at Sta. 100+00 to a 30' Depth at Sta. 101+00.
2. Provide Contact of Pervious Drain with Downstream Pervious in Cutoff.
3. Work by Prior Contracts (Cross Hatched).

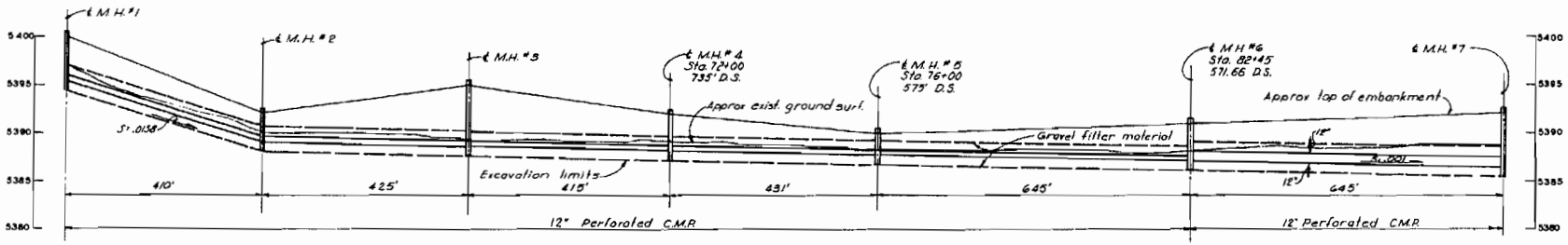
SCALE 1 INCH = 50 FEET

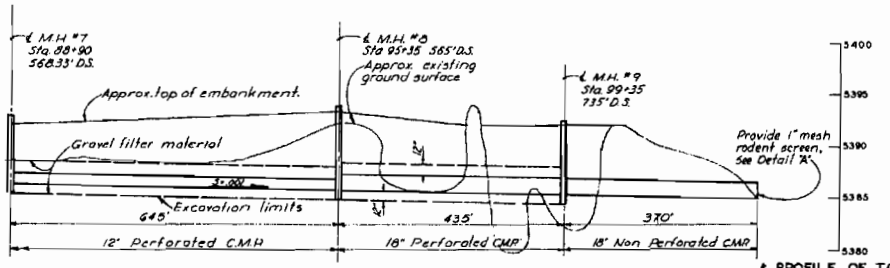
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EARTHWORK - STAGE III
TYPICAL EMBANKMENT SECTIONS
SHEET 3

DESIGNED BY: R. J. M.
DRAWN BY: D. A.
TRACED BY:
CHECKED BY: L. G. S.
SUBMITTED BY: Chief, Design Branch
APPROVED: Chief, Foundations Branch
APPROVED: Charles L. Hipp, Chief, Engineering Division
DATE: JAN. 1970
APPROVED: B. P. Pendergrass, Col., C. E., District Engineer
SCALE:
SPEC. NO. DACW45-70-B-0056
DRAWING NUMBER
SHEET PC6-40E16

CHATFIELD RE-ALLOCATION STUDY PLATE A-10

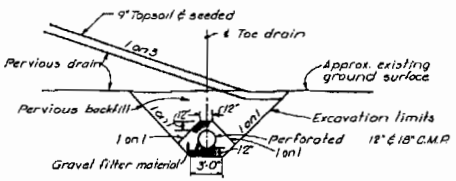




℄ PROFILE OF TOE DRAIN

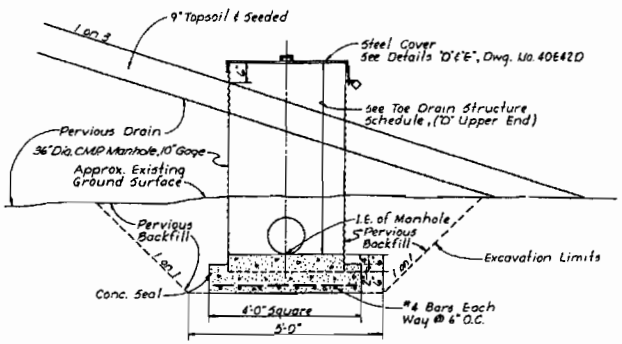
SCALE VERT. 1" = 5' HORIZ. 1" = 100'

TOE DRAIN STRUCTURE SCHEDULE										
From		To		Rim Elev.	Invert Elev.		Slope	℄ to ℄	"D" Upper	
Type	No.	Type	No.	Upper End	From	To	FL/FL	Distance	End	
MH	1	MH	2	5400.5	5395.50	5389.02	.0158	410.00'	5.00'	
MH	2	MH	3	5392.5	5389.02	5388.59	.001	425.00'	3.48'	
MH	3	MH	4	5395.5	5388.59	5388.18	.001	415.00'	6.91'	
MH	4	MH	5	5392.5	5388.18	5387.74	.001	431.00'	4.32'	
MH	5	MH	6	5390.5	5387.74	5387.10	.001	645.00'	2.76'	
MH	6	MH	7	5391.6	5387.10	5386.45	.001	645.00'	4.50'	
MH	7	MH	8	5392.7	5386.45	5385.81	.001	645.00'	6.25'	
MH	8	MH	9	5393.8	5385.81	5385.37	.001	435.00'	7.99'	
MH	9	Outlet		5392.5	5385.37	5385.00	.001	370.00'	7.13'	



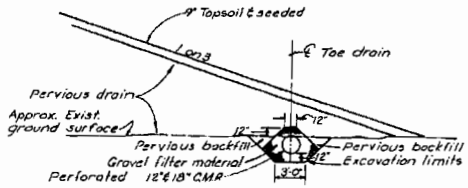
TYPICAL SECTION OUTSIDE OF EMBANKMENT TOE

SCALE: 1 INCH = 5 FEET



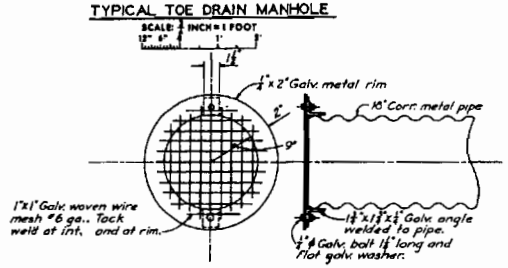
TYPICAL TOE DRAIN MANHOLE

SCALE: 1/4 INCH = 1 FOOT



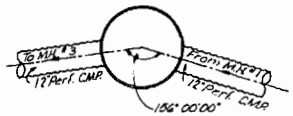
TYPICAL SECTION INSIDE OF EMBANKMENT TOE

SCALE: 1 INCH = 5 FEET

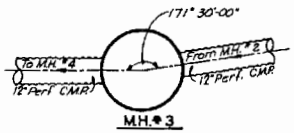


RODENT SCREEN DETAIL (A)

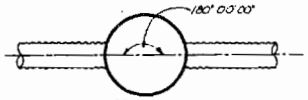
NO SCALE



M.H. #2



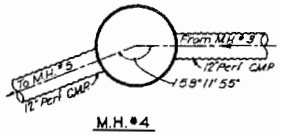
M.H. #3



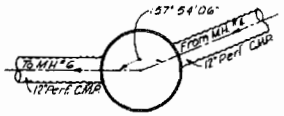
M.H. #6 & #7

TOE DRAIN MANHOLE LAYOUTS

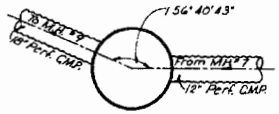
NO SCALE



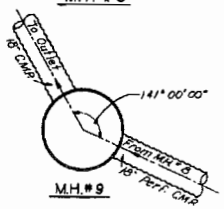
M.H. #4



M.H. #5



M.H. #8



M.H. #9

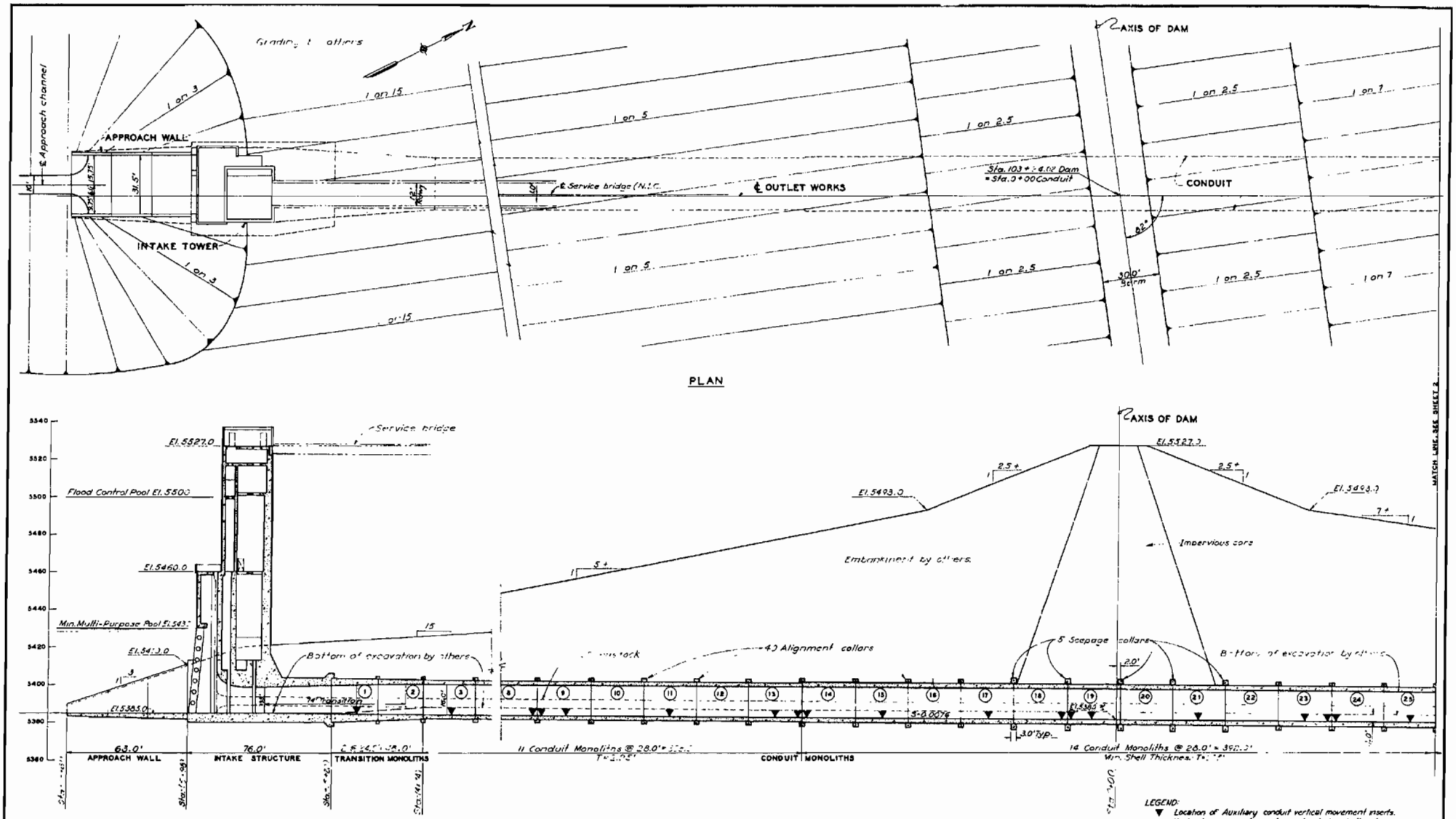
NOTES:

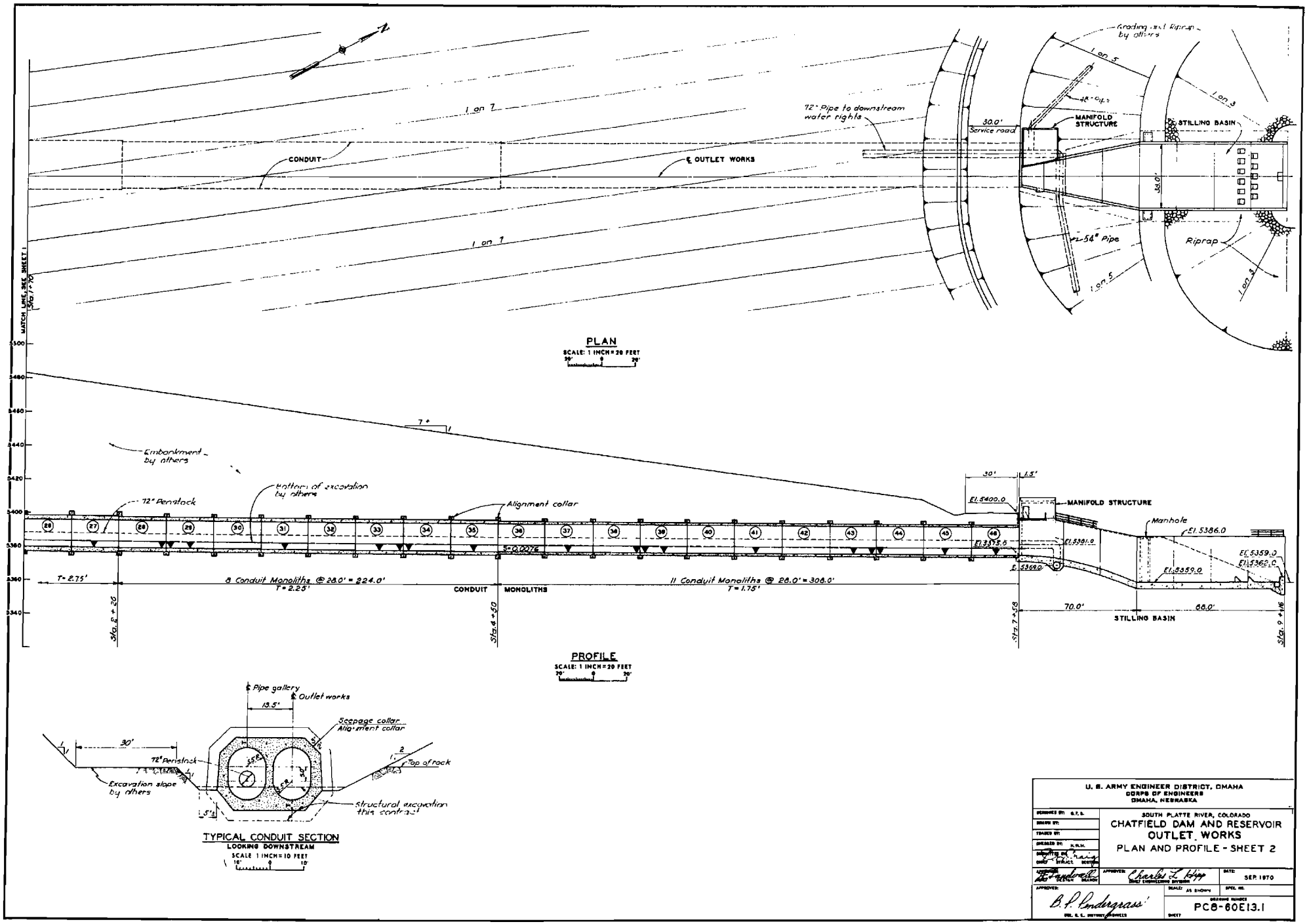
1. For location of toe drain see dwgs. 40E09, 10 & 11.

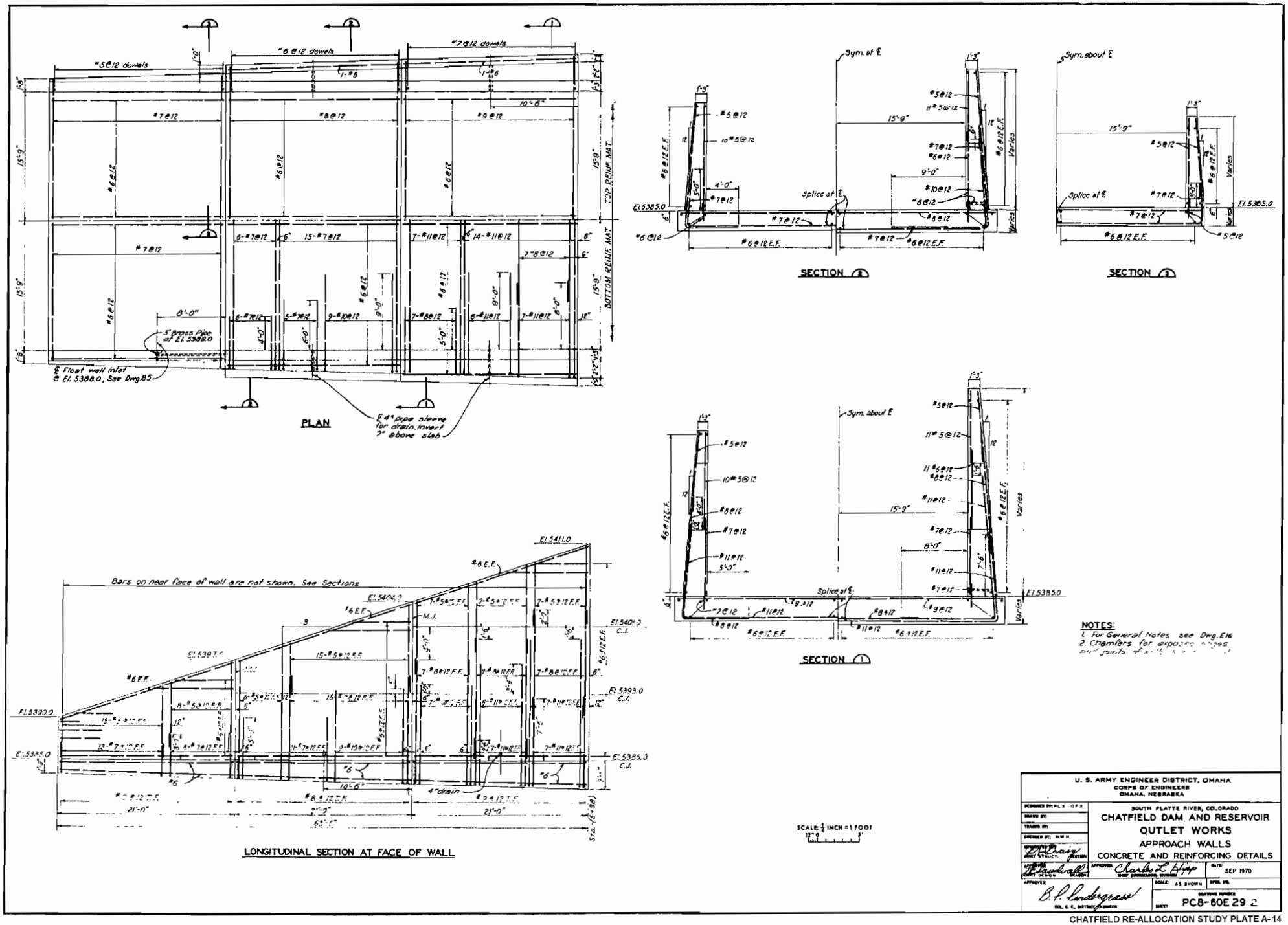
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

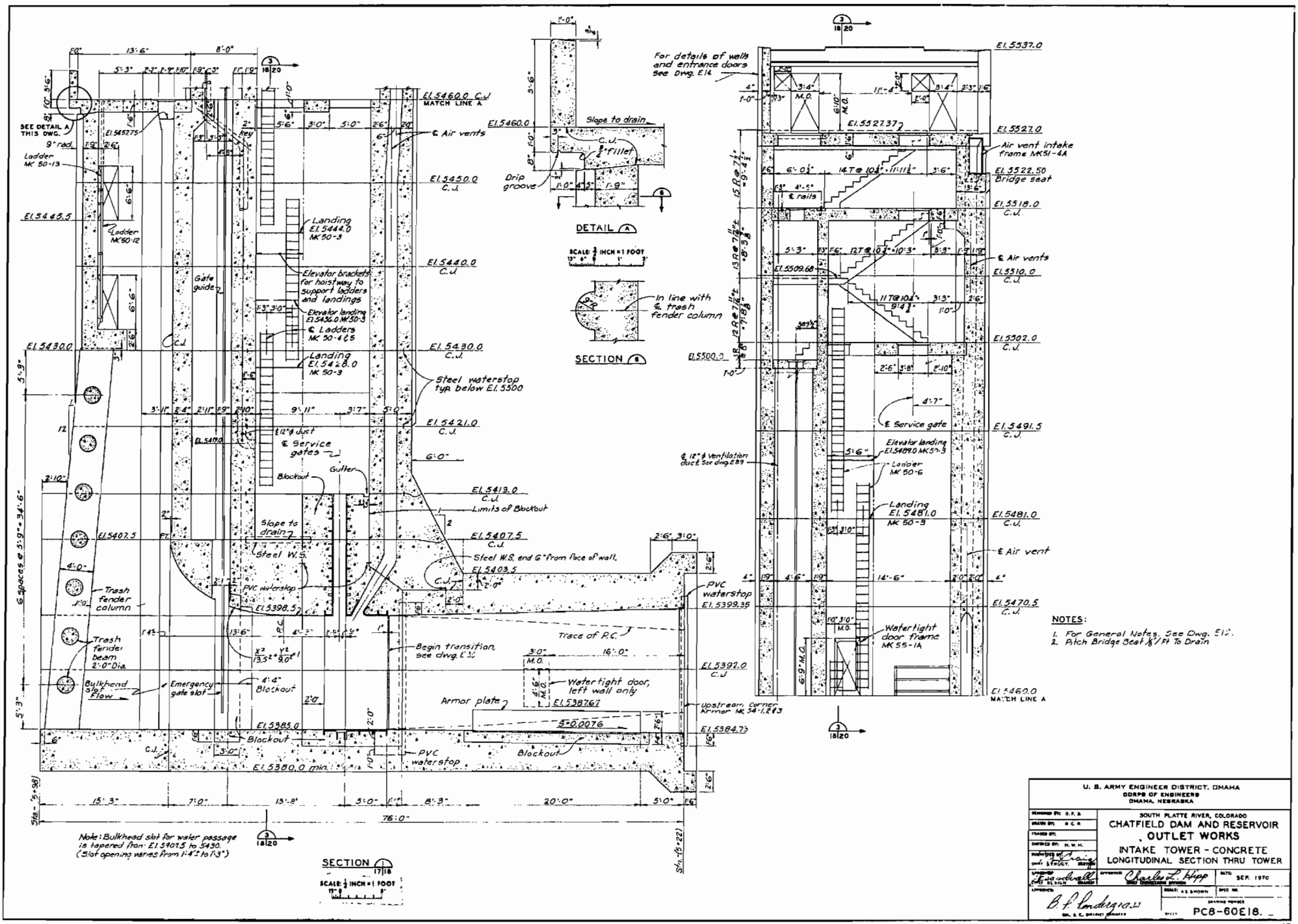
SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EARTHWORK – STAGE III
TOE DRAIN PROFILE & DETAILS

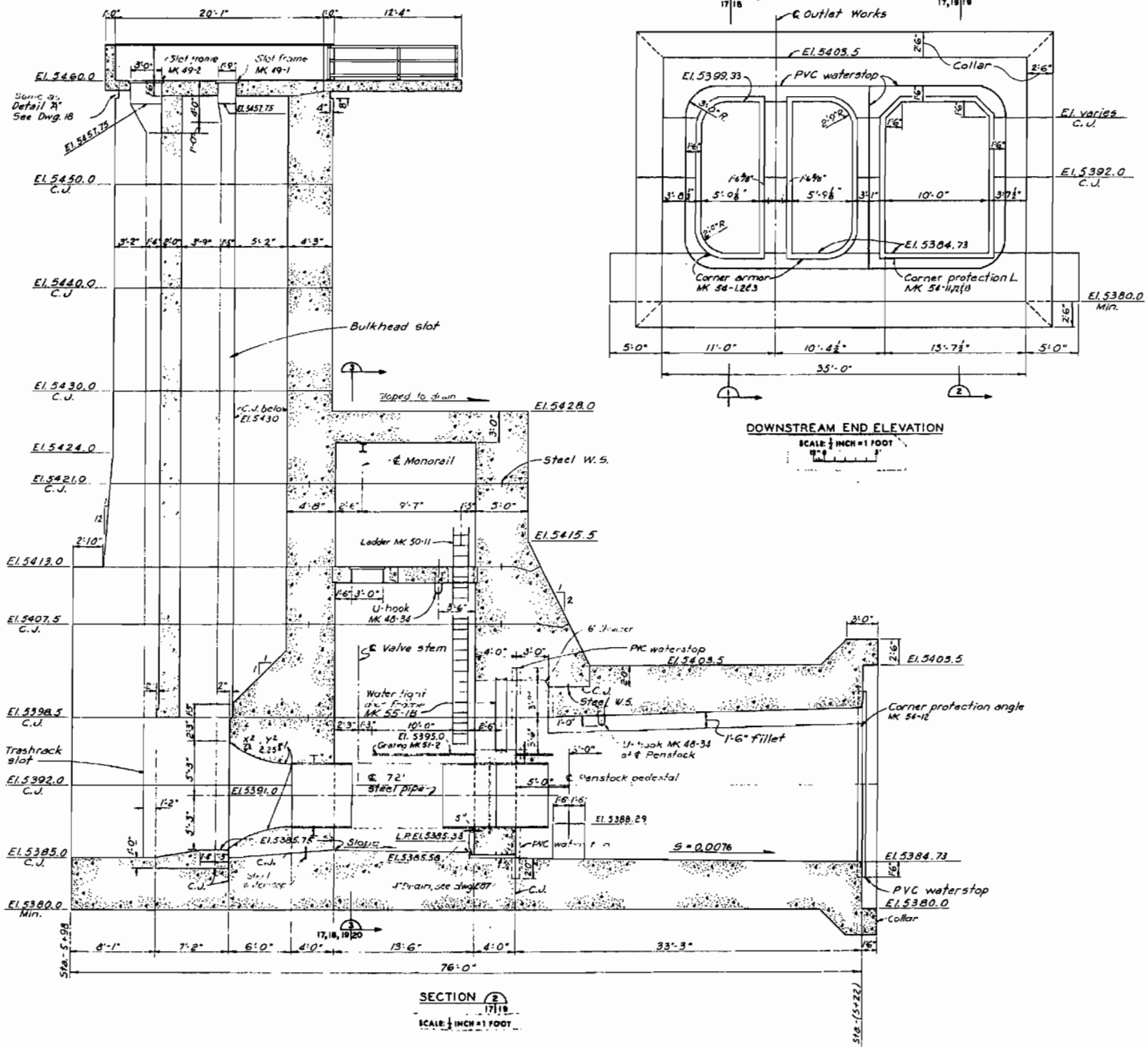
DESIGNED BY: R. J. MB
DRAWN BY: R. G. R.
TRACED BY:
CHECKED BY: C. V. J.
APPROVED: Chas. L. Hipp
DATE: JAN. 1970
SCALE: AS SHOWN
SPEC. NO. DACW45-70-B-0056
DRAWING NUMBER: PC6-40E43
SHEET

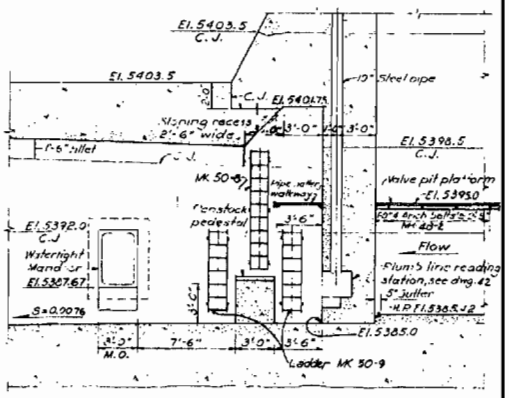

Grading by others
N
℄ Approach channel
APPROACH WALL
INTAKE TOWER
1 on 3
1 on 15
1 on 5
1 on 2.5
1 on 7
℄ Service bridge (N.I.C.)
℄ OUTLET WORKS
Sta. 103+14.02 Dam
= Sta. 0+00 Conduit
CONDUIT
AXIS OF DAM
82°
300' Skirm
PLAN
Service bridge
El. 5527.0
Flood Control Pool El. 5500
El. 5460.0
Min. Multi-Purpose Pool El. 5433
El. 5433.0
El. 5383.0
Bottom of excavation by others
Transition
Embankment by others
Impervious core
El. 5493.0
2.5 +
5 +
7 +
4) Alignment collars
5) Seepage collars
Bottom of excavation by others
El. 5383.0
S=0.0076
30' Typ.
20'
63.0'
APPROACH WALL
76.0'
INTAKE STRUCTURE
TRANSITION MONOLITHS
11 Conduit Monoliths @ 28.0' = 308.0'
CONDUIT MONOLITHS
14 Conduit Monoliths @ 28.0' = 392.0'
Min. Shell Thickness
PROFILE
MATCH LINE, SEE SHEET 2
LEGEND:
▼ Location of Auxiliary conduit vertical movement inserts. Vertical movement inserts are located on both sides of the joint between monoliths 3 and 4 and at the centerlines of monoliths 5 and 7.
SCALE: 1 INCH = 20 FEET
30' 0 30'
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
OUTLET WORKS
PLAN AND PLOT - SHEET 1
DESIGNED BY: G. F. S.
DRAWN BY:
TRACED BY:
CHECKED BY: H. W. H.
CHIEF STRUCT. DESIGN
CHIEF DESIGN BRANCH
APPROVED: Charles L. Higgs
CHIEF ENGINEERING DIVISION
DATE: SEP. 1970
APPROVED: B. P. Pendergrass
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
PC8-60E12
SHEET


PLAN
SCALE: 1 INCH = 20 FEET
1 on 7
CONDUIT
℄ OUTLET WORKS
72" Pipe to downstream water rights
30.0' Service road
MANIFOLD STRUCTURE
STILLING BASIN
Grading and Riprap by others
1 on 5
1 on 3
48" Pipe
54" Pipe
Riprap
33.0'
MATCH LINE, SEE SHEET 1
Sta. 1+70
PROFILE
SCALE: 1 INCH = 20 FEET
Embankment by others
Bottom of excavation by others
72" Penstock
Alignment collar
S=0.0076
El. 5400.0
El. 5391.0
El. 5383.0
El. 5369.0
El. 5386.0
El. 5359.0
El. 5360.0
Manhole
T= 2.75'
8 Conduit Monoliths @ 28.0' = 224.0'
T= 2.25'
CONDUIT MONOLITHS
11 Conduit Monoliths @ 28.0' = 308.0'
T= 1.75'
70.0'
66.0'
STILLING BASIN
Sta. 2+26
Sta. 4+50
Sta. 7+58
Sta. 9+16
TYPICAL CONDUIT SECTION
LOOKING DOWNSTREAM
SCALE 1 INCH = 10 FEET
℄ Pipe gallery
℄ Outlet works
13.5'
Seepage collar
Alignment collar
72" Penstock
Top of rock
Excavation slope by others
Structural excavation this contract
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
OUTLET WORKS
PLAN AND PROFILE - SHEET 2
SEP. 1970
PCB-60E13.1


PLAN
SECTION 2
SECTION 3
SECTION 1
LONGITUDINAL SECTION AT FACE OF WALL
Bars on near face of wall are not shown. See Sections
TOP REINF. MAT.
BOTTOM REINF. MAT.
℄ Float well inlet @ El. 5388.0, See Dwg. 85
℄ 4" pipe sleeve for drain, invert 7" above slab
5" brass Pipe at El. 5388.0
Sym. about ℄
Splice at ℄
El. 5385.0
El. 5411.0
El. 5397.0
El. 5393.0
El. 5385.0 C.J.
4" drain
SCALE: 1/4 INCH = 1 FOOT
NOTES:
1. For General Notes see Dwg. E16
2. Chambers for exposed joints and joints of
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM, AND RESERVOIR
OUTLET WORKS
APPROACH WALLS
CONCRETE AND REINFORCING DETAILS
SEP 1970
AS SHOWN
PC8-60E 29 2


SEE DETAIL A THIS DWG.
9" rad.
Ladder MK 50-13
Ladder MK 50-12
Gate guide
Landing El. 5444.0 MK 50-3
Elevator brackets for hoistway to support ladders and landings
Elevator landing El. 5436.0 MK 50-3
℄ Ladders MK 50-4 & 5
Landing El. 5428.0 MK 50-3
Steel waterstop typ. below El. 5500
12" ϕ duct
℄ Service gates
Gutter
Blockout
Limits of Blockout
Slope to drain
Steel W.S.
Steel W.S. and 6" from face of wall
PVC waterstop
Trash fender column
Trash fender beam 2'-0" Dia.
Bulkhead slot
Flow
Emergency gate slot
4'-4" Blockout
Begin transition, see dwg. E30
Trace of P.C.
Water tight door, left wall only
Armor plate
Upstream corner Armor MK 54-1.2&3
PVC waterstop
Blockout
6 Spaces @ 5'-9" = 34'-6"
El. 5460.0 C.J. MATCH LINE A
El. 5450.0 C.J.
El. 5445.5
El. 5440.0 C.J.
El. 5430.0
El. 5421.0 C.J.
El. 5413.0 C.J.
El. 5407.5
El. 5403.5
El. 5398.5
El. 5392.0 C.J.
El. 5385.0
El. 5380.0 min.
El. 5399.35
El. 5384.73
El. 538767
5'-0.0076
76'-0"
Sta. 15+98.1
Sta. 15+22.1
DETAIL A
SCALE: 1/2 INCH = 1 FOOT
Slope to drain
Drip groove
In line with ℄ trash fender column
SECTION B
SECTION
SCALE: 1/2 INCH = 1 FOOT
For details of walls and entrance doors see Dwg. E14
El. 5537.0
El. 5527.0
Air vent intake frame MK51-4A
El. 5522.50 Bridge seat
El. 5518.0 C.J.
℄ Air vents
El. 5510.0 C.J.
El. 5502.0 C.J.
℄ Service gate
El. 5491.5 C.J.
Elevator landing El. 5489.0 MK 50-3
℄ 12" ϕ ventilation duct, See dwg. E89
Ladder MK 50-6
Landing El. 5481.0 MK 50-3
El. 5481.0 C.J.
℄ Air vent
El. 5470.5 C.J.
Water tight door frame MK 55-1A
El. 5460.0 MATCH LINE A
El. 5500.0
NOTES:
1. For General Notes, See Dwg. E12.
2. Pitch Bridge Seat 1/8"/Ft To Drain
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
OUTLET WORKS
INTAKE TOWER - CONCRETE
LONGITUDINAL SECTION THRU TOWER
DESIGNED BY: O.F.B.
DRAWN BY: R.C.A.
CHECKED BY: H.W.H.
SUBMITTED BY: Charles L. Hipp
DATE: SEP. 1970
SCALE: AS SHOWN
DRAWING NUMBER
PC8-60E18.





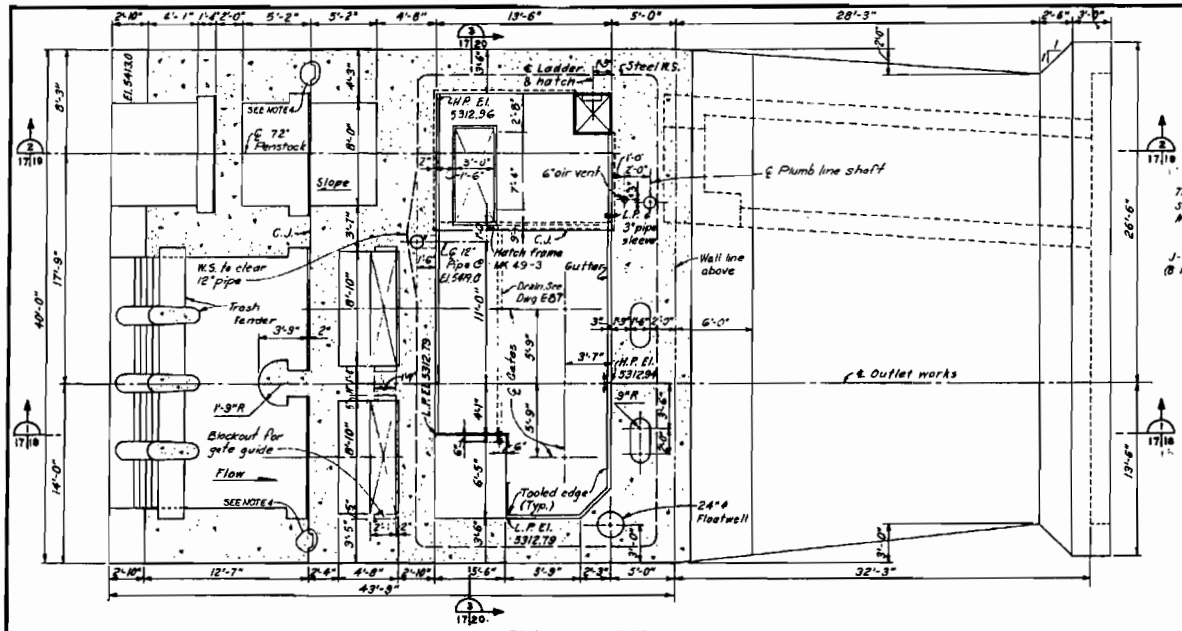
NOTES:

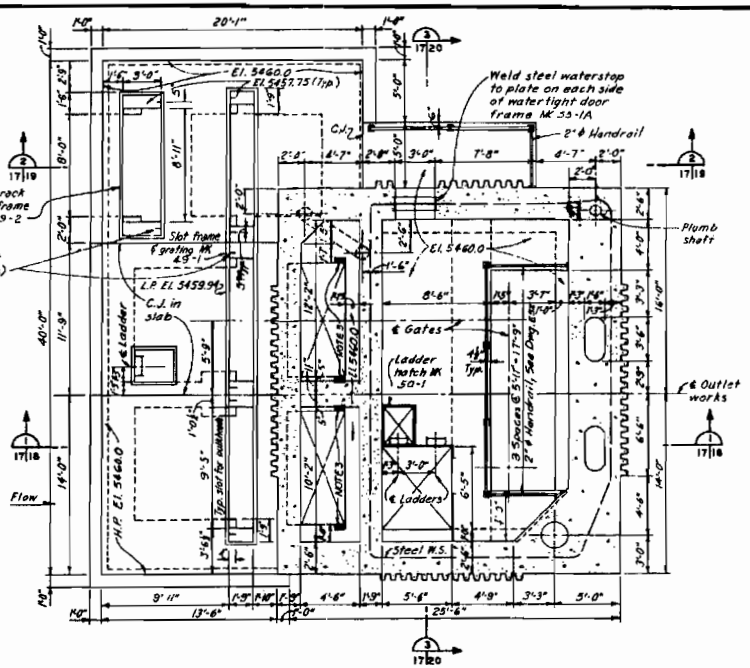
For General Notes, See Dwg. E16.

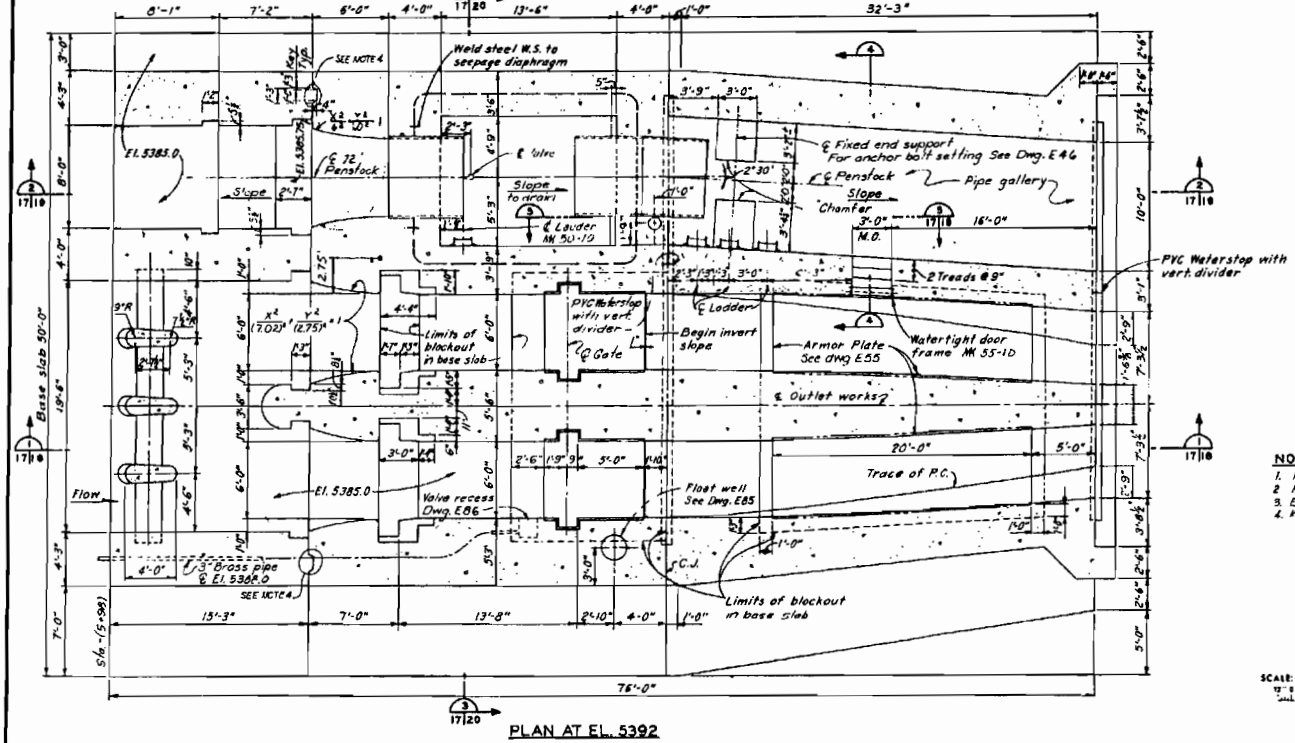
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

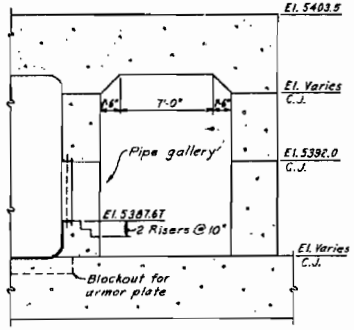
SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
OUTLET WORKS
INTAKE TOWER - CONCRETE
SECTIONS THRU PIPE GALLERY

DESIGNED BY: P.L.B. D.F.A.
DRAWN BY: R.C.K.
TRACED BY:
CHECKED BY: H.W.H.
CHIEF STRUCT. DESIGN:
CHIEF DESIGN:
APPROVED: Charles L. Hipp
CHIEF, ENGINEERING DIVISION
DATE: SEP 1970
SCALE: AS SHOWN
APPROVED: B. P. Pendergrass
DIR. C. L. DISTRICT ENGINEER
DRAWING NUMBER: PC8-60E192
SHEET:









SECTION 4

NOTES:

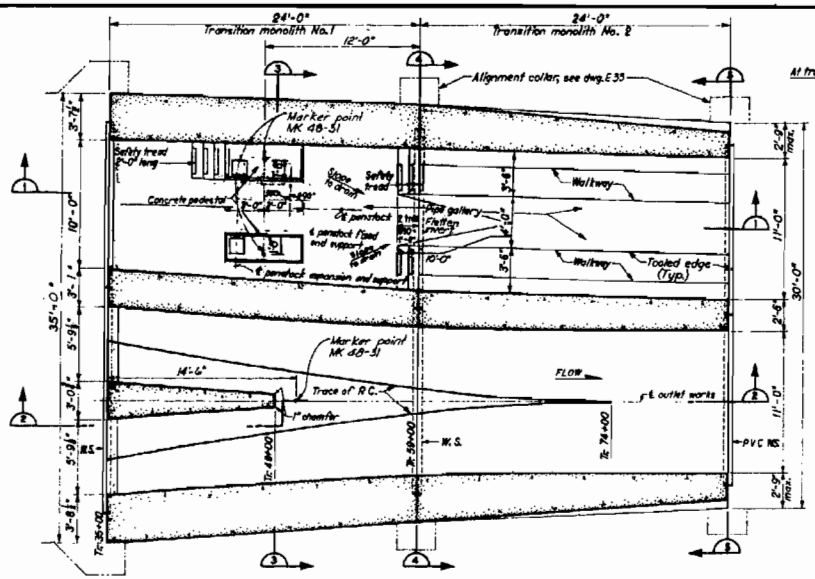
1. For General Notes, see dwg. E1
2. For Details of J-bolt, see dwg. E[illegible]
3. Emergency blockout El 5460 [illegible]
4. Keyways in outside walls enter[illegible]

SCALE: 1/8 INCH = 1 FOOT

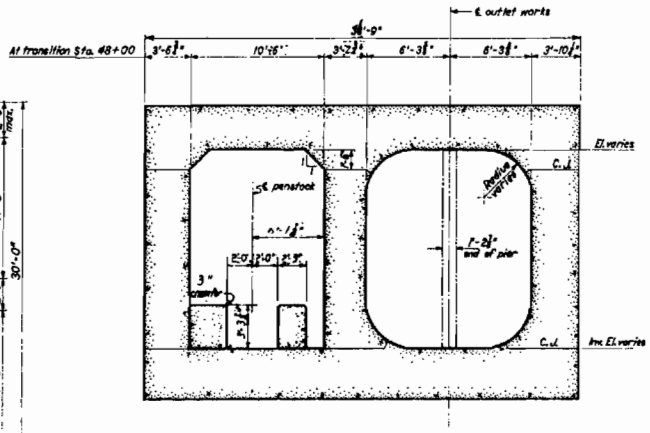
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
OUTLET WORKS
INTAKE TOWER - CONCRETE
LOWER SECTIONAL PLANS

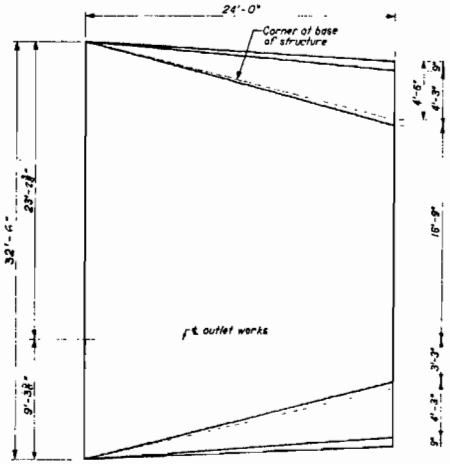
DESIGNED BY: G. F. G.
DRAWN BY: R. W. W.
TRACED BY:
CHECKED BY: H. W. H.
SUBMITTED BY:
CHIEF STRUCT.
APPROVED: Charles L. Hipp, CHIEF, ENGINEERING DIVISION
DATE: SEP. 1970
SCALE: AS SHOWN
SPEC. NO.
APPROVED: C. F. Pendergrass, COL., C. E., DISTRICT ENGINEER
DRAWING NUMBER: PC8-60E17.2
SHEET



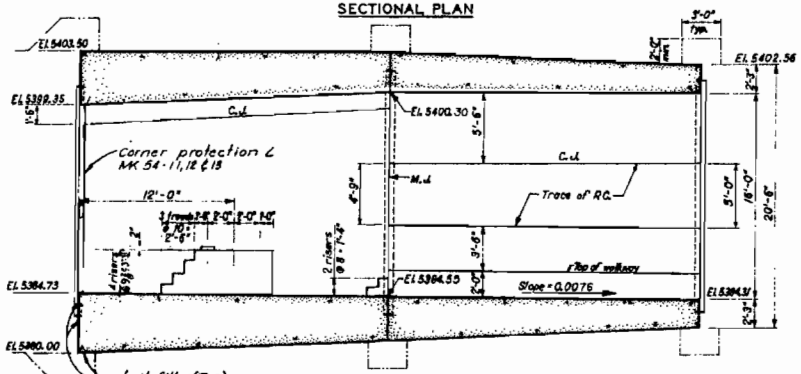
SECTIONAL PLAN



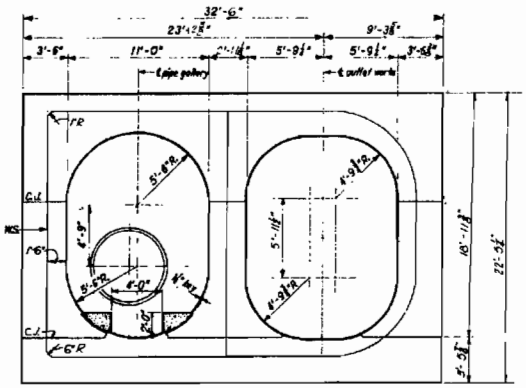
SECTION 3



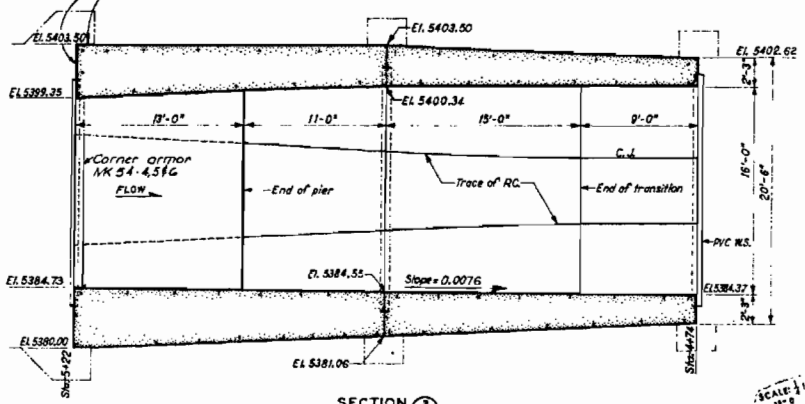
PLAN - MONOLITH 2



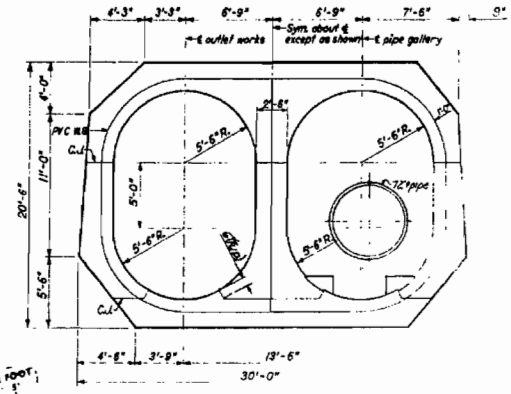
SECTION 1



SECTION 4



SECTION 2



SECTION 5

SCALE: ½ INCH = 1 FOOT

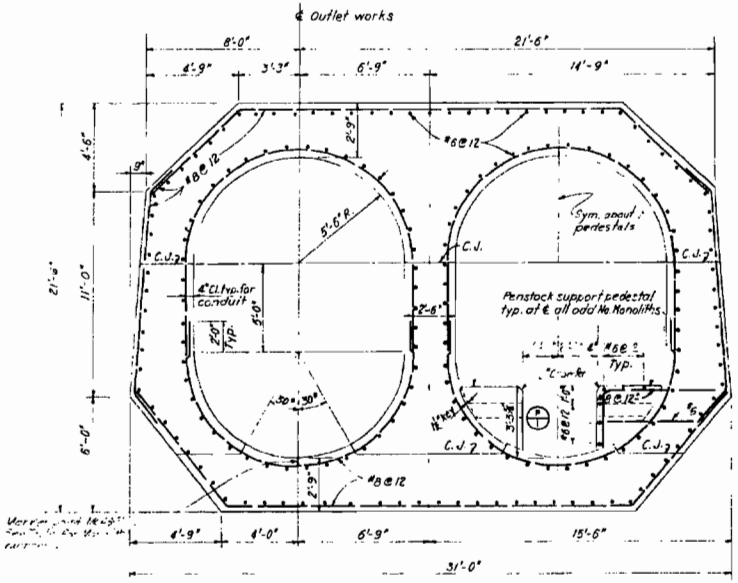
NOTE:

1. For General Notes, see dwg. E16
2. Joint filler shall cover full width of contact areas on each side of waterstop.

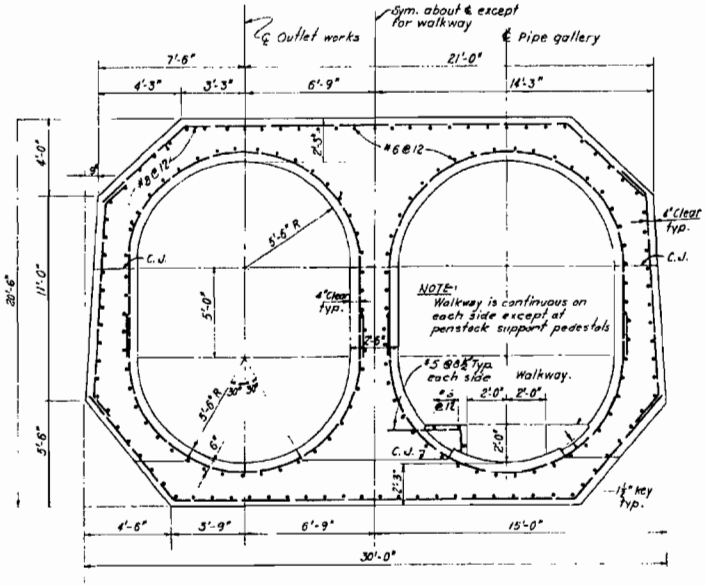
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER COLORADO
CHATFIELD DAM AND RESERVOIR
OUTLET WORKS
TRANSITION MONOLITHS
CONCRETE DETAILS

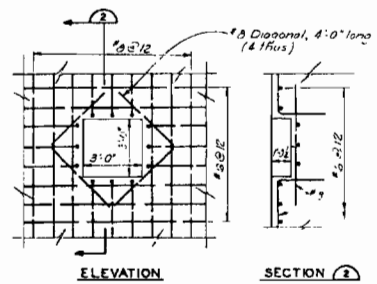
DESIGNED BY: O.F.J.
DRAWN BY: R.F.J.
TRACED BY:
CHECKED BY: H.W.H.
SUBMITTED BY: J. P. Kelly, CHIEF STRUCT. BRANCH
APPROVED: Charles L. Hipp, CHIEF ENGINEERING DIVISION
APPROVED: B. P. Vandergraas, COL. C. E. DISTRICT ENGINEER
DATE: SEP. 1970
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER: PC8-60E31.2
SHEET



MAXIMUM CONDUIT SECTION T=2'-9"
MONOLITHS 14 THRU 27



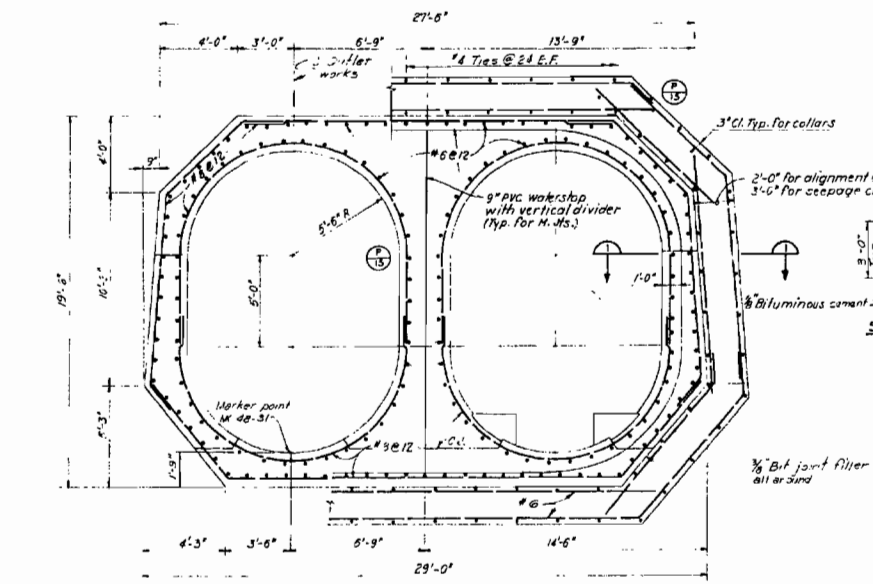
INTERMEDIATE CONDUIT SECTION T=2'-3"
MONOLITHS 3 THRU 13 AND 28 THUU 35



TERMINAL BOX RECESS

Note:
Recess are located in monoliths 8 & 11 ER
see dwg. E44 for details

Note:
All conduit sections shown are looking upstream



MINIMUM CONDUIT SECTION T=1'-9"
AND TYP. COLLAR DETAILS
MONOLITHS 36 THRU 46

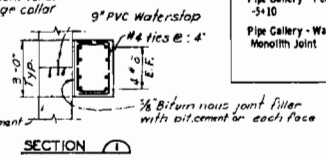


TABLE FOR MARKER POINTS	
LOCATION IN MONOLITH	MONOLITH NOS.
Outlet Conduit - Invert at mid-point of monolith.	4, 6, 8, 10, 12, 14, 16, 18, 19, 20, 21, 23, 25, 27, 29, 31, 33, 35, 37, 39, 41, 43, 45
Transition - Invert Station -5+7.5	1
Pipe Gallery - Pedestals at mid-point of monolith.	3, 5, 7, 9, 11, 13, 15, 17, 19, 21, 23, 25, 27, 29, 31, 33, 35, 37, 39, 41, 43, 45
Pipe Gallery - Pedestals at Station 7+56.17	46
Pipe Gallery - Pedestals at Station -5+10	1
Pipe Gallery - Walkway, 6" each side of Monolith Joint	3 and 4, 8 and 9, 13 and 14, 18 and 19, 23 and 24, 28 and 29, 33 and 34, 38 and 39, 43 and 44

NOTES:
1. For General Notes see Dwg. E12
2. For location of conduit monoliths see Dwgs. E12 & E13

DATE	DESCRIPTION	MADE	APPR'D
8/13/79	REVISED TO SHOW "AS-BUILT" CONDITIONS.	G.C.S.	J.J.G.
P-15 2-7-71	Deleted C.J. at top of collar	E.J.G.	J.J.G.
P-1 1-6-71	Revised Sheet 4 of 4 at Penstock Support Pedestal	[illegible]	[illegible]
Am-1 10-1-70	Revised encircled items in accordance with written description Drawing not issued with this amendment	[illegible]	[illegible]

REVISIONS

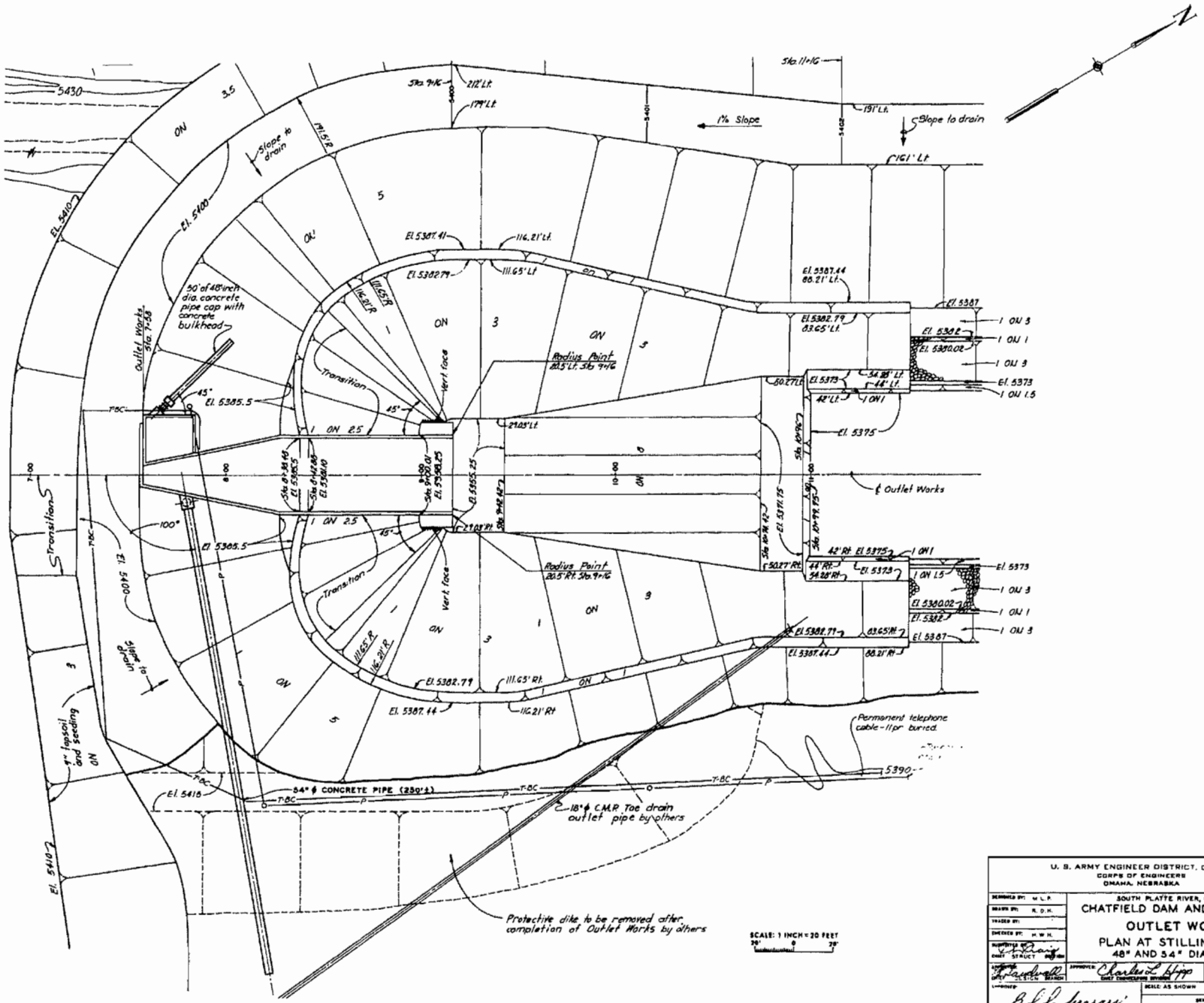
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
OUTLET WORKS
CONDUITS
CONCRETE AND REINFORCING DETAILS

DESIGNED BY: L.R.B.-O.T.N.
DRAWN BY: S.A.
TRACED BY:
CHECKED BY: H.W.H.
APPROVED: Charles L. Hipp, CHIEF, ENGINEERING DIVISION
DATE: SEP. 1970
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER: PCB-60E33.2 (P-15)
SHEET
APPROVED: B. P. Lindgren, SR. R. E. INSPECTOR

SCALE: ½ INCH = 1 FOOT

THIS PLAN ACCOMPANIES CONTRACT No. DACW 45-71-C-0058 MODIFICATION No. P 001

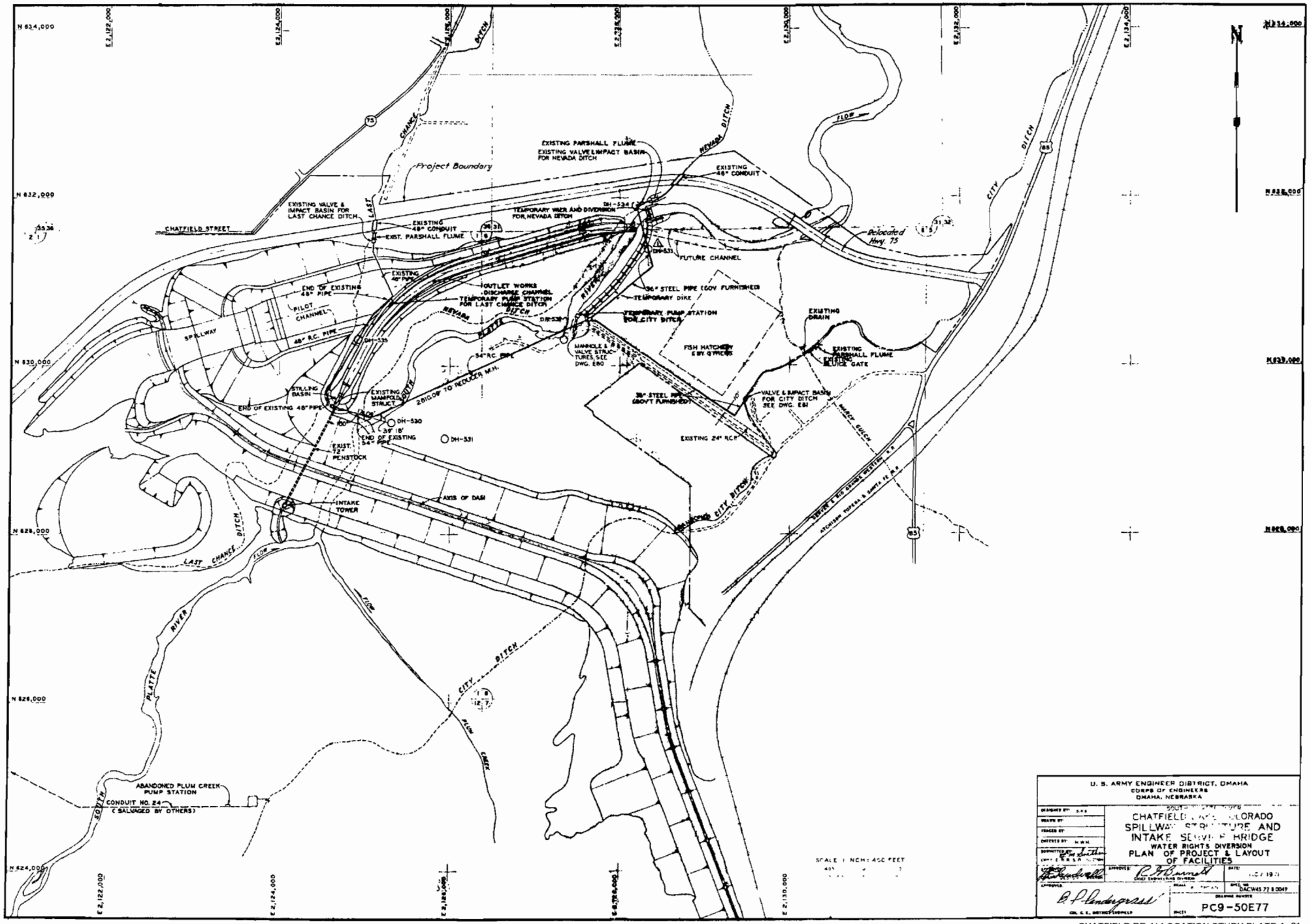


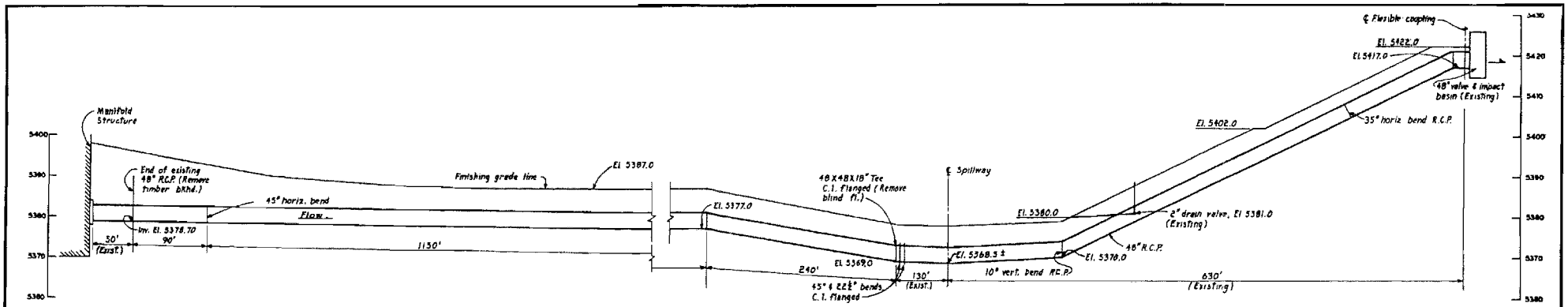
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: M. L. P.
DRAWN BY: R. D. H.
TRACED BY:
CHECKED BY: H. W. H.
SUBMITTED: CHIEF, STRUCT. DESIGN
CHIEF, DESIGN BRANCH
APPROVED: Charles L. Hipp, CHIEF, ENGINEERING DIVISION
LANDRUM: B. F. Landrum, COL., C. E., DISTRICT ENGINEER

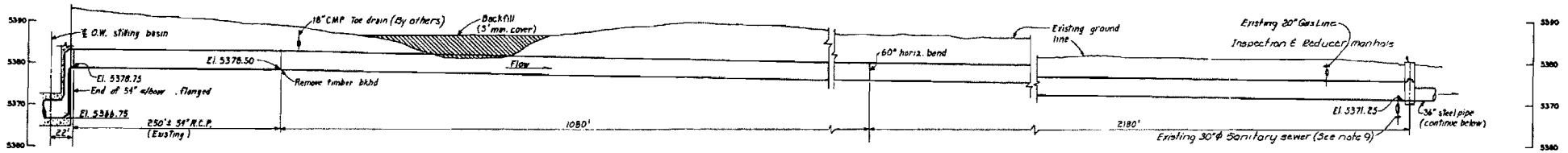
SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
OUTLET WORKS
PLAN AT STILLING BASIN
48" AND 54" DIA. PIPES

DATE: SEP. 1970
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET PC8-60E40.3

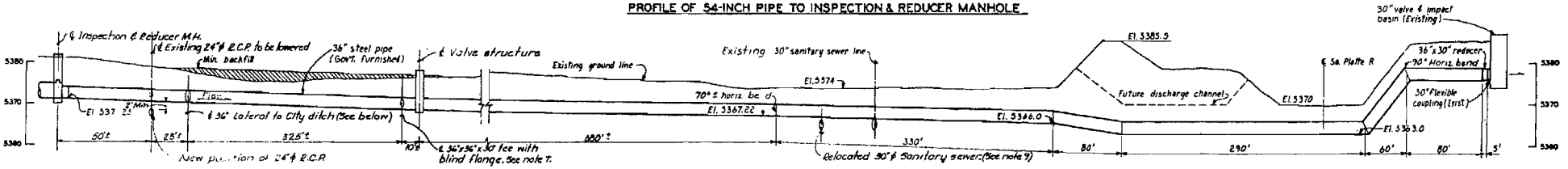

N 834,000
N 832,000
N 830,000
N 828,000
N 826,000
N 824,000
E 2,122,000
E 2,124,000
E 2,126,000
E 2,128,000
E 2,130,000
E 2,132,000
E 2,134,000
N
Project Boundary
CHATFIELD STREET
EXISTING VALVE & IMPACT BASIN FOR LAST CHANCE DITCH
EXISTING 48" CONDUIT
EXIST. PARSHALL FLUME
EXISTING PARSHALL FLUME
EXISTING VALVE & IMPACT BASIN FOR NEVADA DITCH
TEMPORARY WEIR AND DIVERSION FOR NEVADA DITCH
EXISTING 48" CONDUIT
Relocated Hwy. 75
FUTURE CHANNEL
36" STEEL PIPE (GOV FURNISHED)
TEMPORARY DIKE
TEMPORARY PUMP STATION FOR CITY DITCH
EXISTING DRAIN
FISH HATCHERY & BY OTHERS
EXISTING PARSHALL FLUME & SLUICE GATE
VALVE & IMPACT BASIN FOR CITY DITCH SEE DWG. E8
EXISTING 24" RCP
MANHOLE & VALVE STRUCTURES, SEE DWG. E80
OUTLET WORKS DISCHARGE CHANNEL
TEMPORARY PUMP STATION FOR LAST CHANCE DITCH
END OF EXISTING 48" PIPE
PILOT CHANNEL
SPILLWAY
48" R.C. PIPE
54" R.C. PIPE
STILLING BASIN
EXISTING MANIFOLD STRUCT.
END OF EXISTING 48" PIPE
DH-530
36" 18'
END OF EXISTING 54" PIPE
DH-531
EXIST. 72" PENSTOCK
INTAKE TOWER
AXIS OF DAM
ABANDONED CITY DITCH
ATCHISON TOPEKA & SANTA FE R.R.
LAST CHANCE DITCH
PLATTE RIVER
CITY DITCH
FLUM CREEK
ABANDONED PLUM CREEK PUMP STATION
CONDUIT NO. 24 (SALVAGED BY OTHERS)
SCALE 1 INCH = 400 FEET
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
CHATFIELD DAM, COLORADO
SPILLWAY STRUCTURE AND
INTAKE SERVICE BRIDGE
WATER RIGHTS DIVERSION
PLAN OF PROJECT & LAYOUT
OF FACILITIES
DACW45 72 B 0049
PC9-50E77



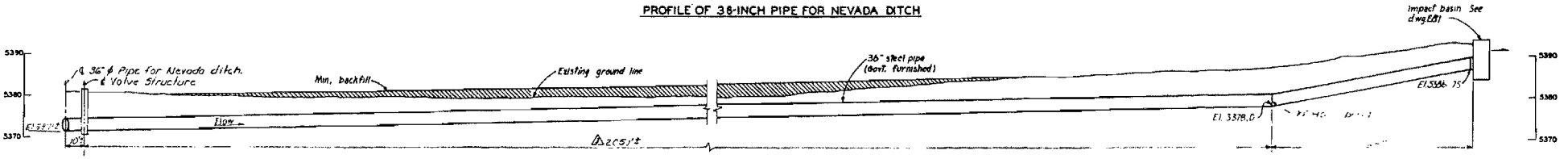
PROFILE OF 48-INCH PIPE FOR LAST CHANCE DITCH



PROFILE OF 54-INCH PIPE TO INSPECTION & REDUCER MANHOLE



PROFILE OF 36-INCH PIPE FOR NEVADA DITCH



PROFILE OF 36-INCH PIPE FOR CITY DITCH

SCALE VERT. 1 INCH = 10 FEET
HORIZ. 1 INCH = 50 FEET

NOTES:

1. Reinforced concrete pipe (R. C. P.) shall conform to ASTM designation C361-70a, class D-125.
2. Pipe joints and gaskets for R. C. P. shall conform to AWWA C302.
3. All connections between steel pipe and R. C. P. shall have approved adapters.
4. 36-inch steel pipe (Government Furnished) is 1/4" wall thickness, 40' lengths, dresser coupled and is coal tar enamel lined and coated.
5. All pipes shall have minimum cover of 5' unless otherwise noted.
6. Lengths and elevations shown are approximations.
7. 36"x36"x30" tee with blind flange shall be located as shown or as directed by the Contracting Officer.
8. Concrete thrust blocks shall be installed at all horizontal bends.
9. Existing sanitary sewer shall be lowered if necessary to provide 2'-0" minimum clearance from the new pipe.

THRUST BLOCK AREAS REQUIRED

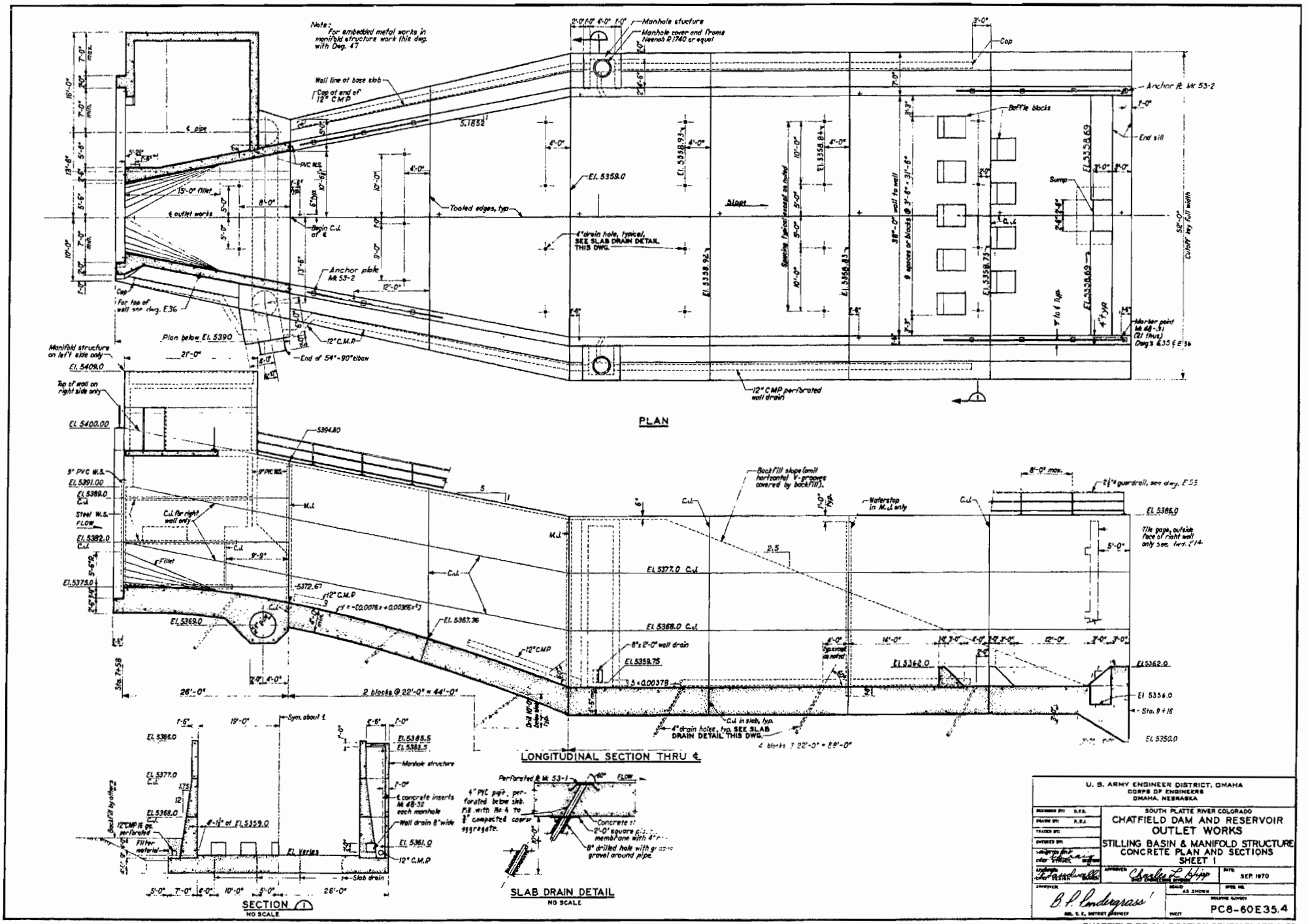
DEGREE BEND	PIPE SIZE	THRUST BLOCK AREA
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]

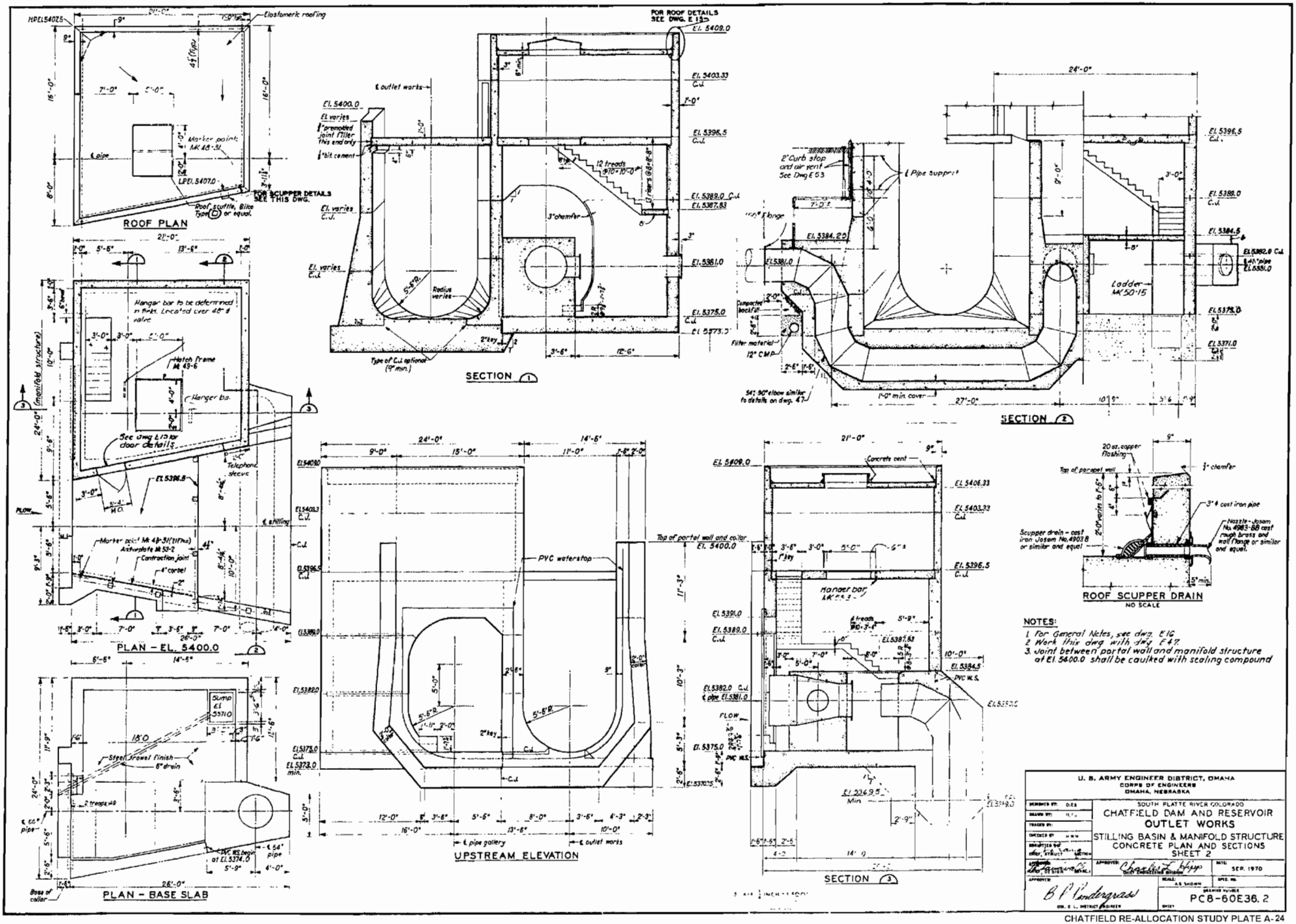
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

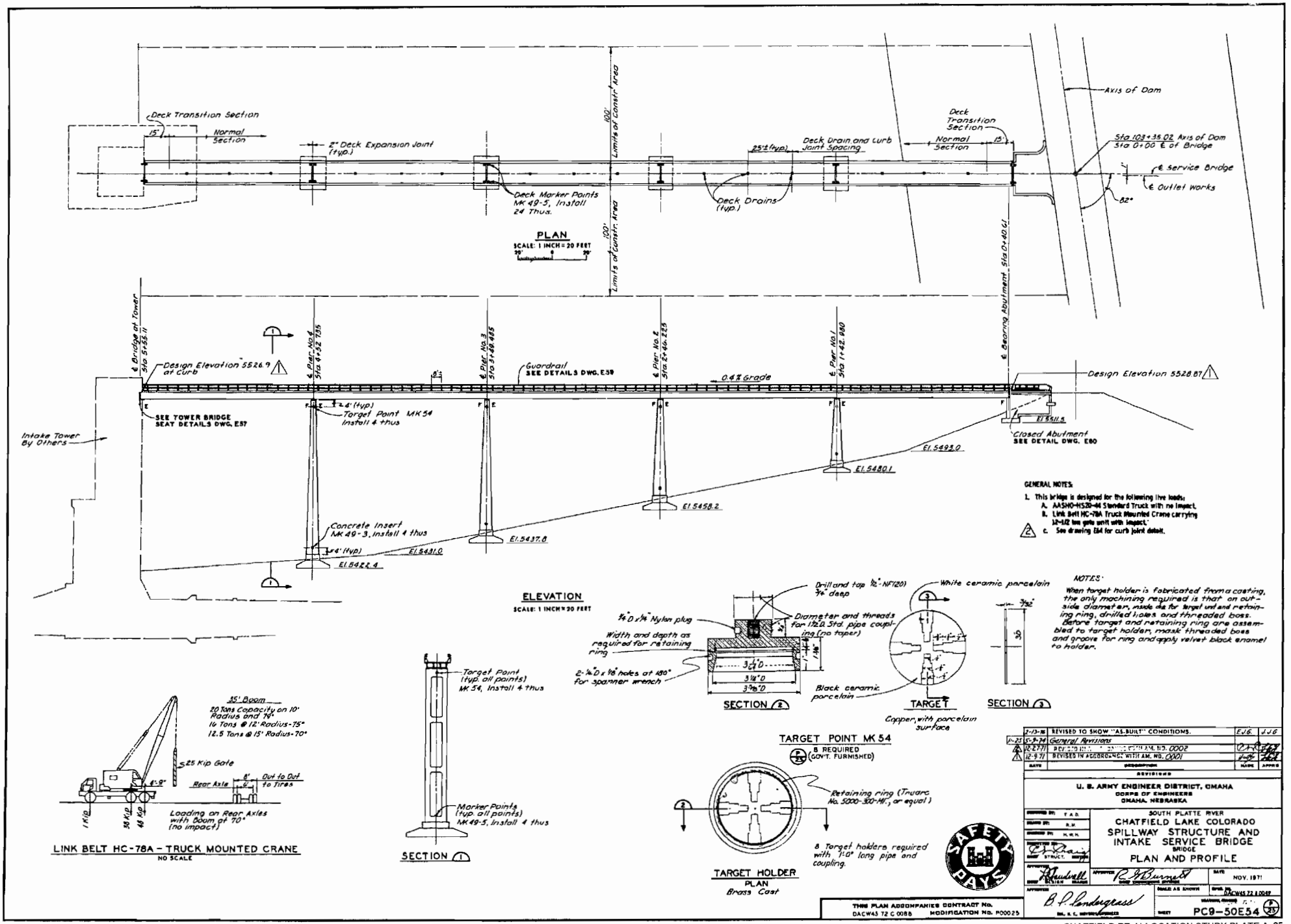
DESIGNED BY: G. F. S.
DRAWN BY: F. J. K.
CHECKED BY: H. W. H.
SUBMITTED BY: [illegible] CHIEF, STRUCT. DESIGN
APPROVED: [illegible] CHIEF, DESIGN BRANCH
APPROVED: [illegible] CHIEF, ENGINEERING DIVISION
APPROVED: [illegible] COL., C. E., DISTRICT ENGINEER

SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
WATER RIGHTS DIVERSION
PIPELINE PROFILES

DATE: NOV 1971
SCALE: AS SHOWN
DWG. NO. DACW45-72-B-0049
DRAWING NUMBER
SHEET PC9-50E79


Note: For embedded metal works in manifold structure work this dwg. with Dwg. 47
Manhole structure
Manhole cover and frame Neenah R 1740 or equal
Cap
Wall line of base slab
Cap at end of 12" CMP
Anchor pl. Mk 53-2
Baffle blocks
End sill
Sump
Tooled edges, typ
4" drain hole, typical, SEE SLAB DRAIN DETAIL THIS DWG.
Slope
Anchor plate Mk 53-2
Cap For top of wall see dwg. E36
Plan below El. 5390
12" C.M.P.
End of 54"-90° elbow
Marker point Mk 48-31 (21 thus) Dwg's E35 & E36
12" CMP perforated wall drain
PLAN
Manifold structure on left side only
Top of wall on right side only
Backfill slope (omit horizontal V-grooves covered by backfill).
Waterstop in M.J. only
2 1/4" guardrail, see dwg. E55
Tile gage, outside face of right wall only see dwg. E14
Steel W.S. FLOW
C.J. for right wall only
Fillet
12" C.M.P.
8"x2'-0" wall drain
4" drain holes, typ. SEE SLAB DRAIN DETAIL THIS DWG.
C.J. in slab, typ.
2 blocks @ 22'-0" = 44'-0"
2 blocks @ 22'-0" = 88'-0"
LONGITUDINAL SECTION THRU ℄
Sym. about ℄
Manhole structure
℄ concrete inserts Mk 48-32 each manhole
Wall drain 8" wide
Backfill by others
Filter material
Slab drain
SECTION 1
NO SCALE
Perforated & Mk 53-1
FLOW
4" PVC pipe, perforated below slab. Fill with No. 4 to 3/4" compacted coarse aggregate.
Concrete slab
2'-0" square pl. membrane with 4" gravel around pipe.
8" drilled hole with gravel around pipe.
SLAB DRAIN DETAIL
NO SCALE
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
SOUTH PLATTE RIVER COLORADO
CHATFIELD DAM AND RESERVOIR
OUTLET WORKS
STILLING BASIN & MANIFOLD STRUCTURE
CONCRETE PLAN AND SECTIONS
SHEET 1
SEP. 1970
AS SHOWN
PC8-60E35.4


ROOF PLAN
SECTION 1
SECTION 2
SECTION 3
PLAN - EL. 5400.0
PLAN - BASE SLAB
UPSTREAM ELEVATION
ROOF SCUPPER DRAIN
NO SCALE
NOTES:
1. For General Notes, see dwg. E16
2. Work this dwg with dwg. E47.
3. Joint between portal wall and manifold structure at El. 5400.0 shall be caulked with sealing compound
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
SOUTH PLATTE RIVER COLORADO
CHATFIELD DAM AND RESERVOIR
OUTLET WORKS
STILLING BASIN & MANIFOLD STRUCTURE
CONCRETE PLAN AND SECTIONS
SHEET 2
SEP. 1970
PC8-60E36.2



NOTES:

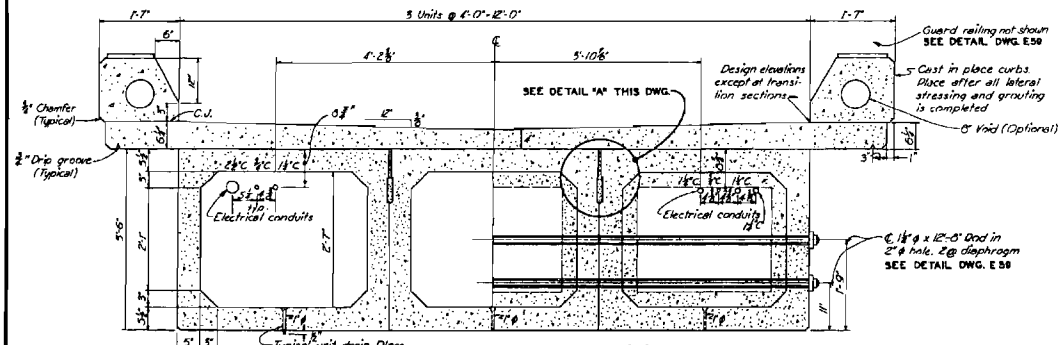
When target holder is fabricated from a casting, the only machining required is that on outside diameter, inside dia for target unit and retaining ring, drilled holes and threaded boss. Before target and retaining ring are assembled to target holder, mask threaded boss and groove for ring and apply velvet black enamel to holder.

GENERAL NOTES:

1. This bridge is designed for the following live loads:
   A. AASHO-HS20-44 Standard Truck with no impact.
   B. Link Belt HC-78A Truck Mounted Crane carrying 12-1/2 ton gate unit with impact.
   C. See drawing E64 for curb joint detail.

	2-13-78	REVISED TO SHOW "AS-BUILT" CONDITIONS.	EJG	JJG
1-25	5-9-74	General Revisions		
2	12-27-71	REVISED IN ACCORDANCE WITH AM. NO. 0002		
1	12-9-71	REVISED IN ACCORDANCE WITH AM. NO. 0001		
DATE		DESCRIPTION	NAME	APPRD

REVISIONS

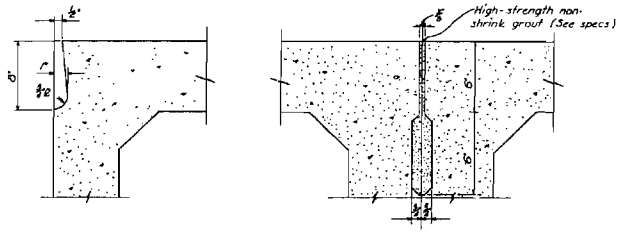


NOTE: After lateral tensioning has been completed fill keys with an approved non-shrink grout. See specs.

NOTE:
1. Use deck section shown above except for 15 Ft. transition sections at each end of bridge.
2. Sections taken looking from tower

HALF SECTION NEAR MIDSPAN

HALF SECTION AT PIER

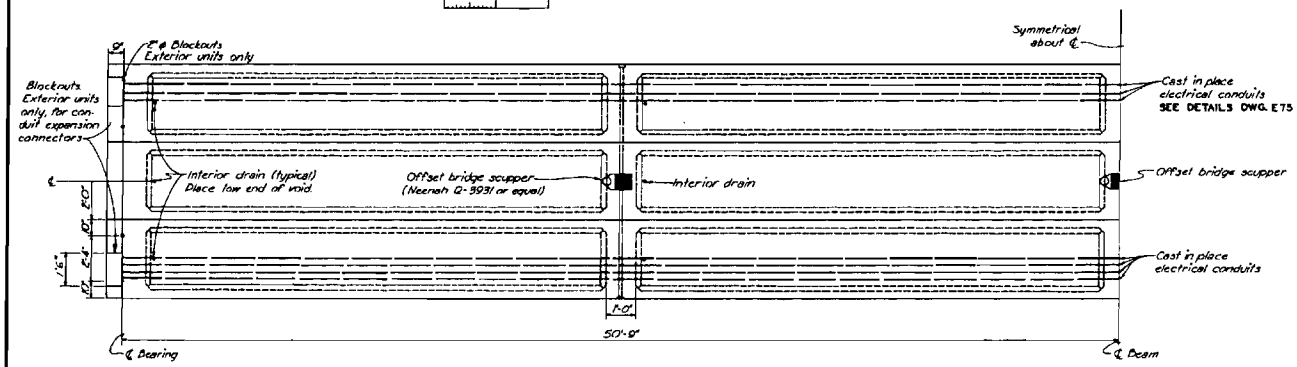


ALTERNATE SHEAR KEY

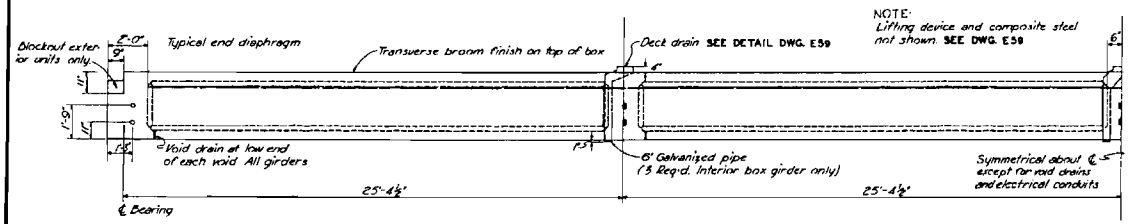
PREFERRED SHEAR KEY

DETAIL A
SCALE: 1"=3'-0"

SCALE: 1 INCH = 1 FOOT



HALF PLAN
TYPICAL 103'-0" UNIT
SCALE: 1/4 INCH = 1 FOOT



NOTE:
Lifting device and composite steel not shown. SEE DWG. E59

LONGITUDINAL ELEVATION INTERIOR BOX

LONGITUDINAL ELEVATION EXTERIOR BOX

HALF ELEVATION
SCALE: 1/4 INCH = 1 FOOT

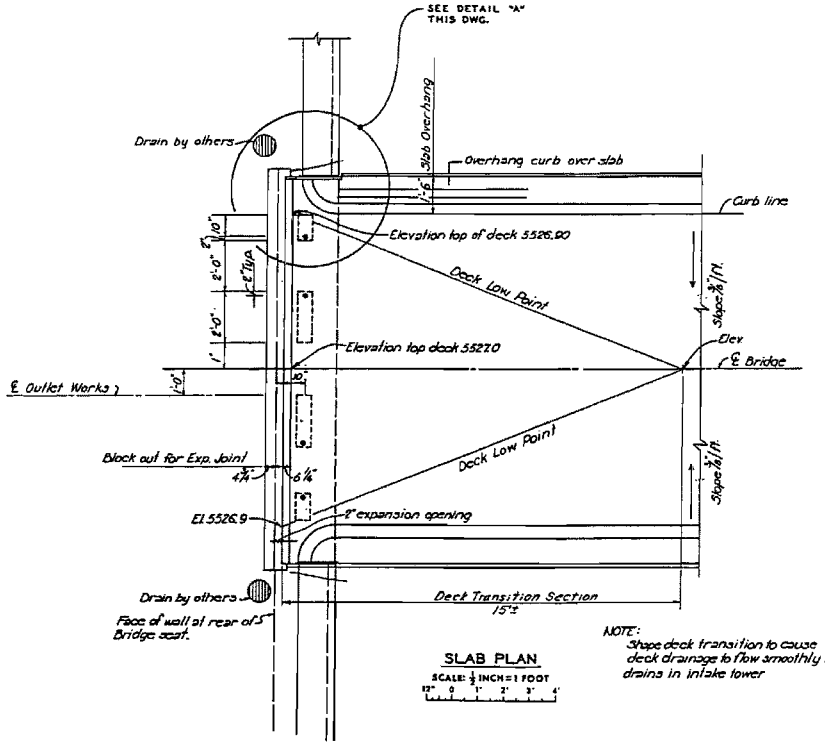
GENERAL NOTES:

1. The precast concrete elements shall be fabricated by a producer regularly engaged in the manufacture of precast concrete products.
2. The section dimensions of the 103 ft. prestressed precast box girders may be altered to accomodate the facilities of the fabricator; however, the basic section shall be maintained and be structurally equivalent. Any alterations shall be subject to the review and approval of the Contracting Officer. See specifications for precast concrete and prestress steel requirements.
3. Lateral Tensioning: Lateral ties shall be provided through the diaphragms in the positions indicated. Each tie shall be equivalent to a 1-1/4" ϕ A36 bar tensioned to 30,000 lbs. Tension in 1-1/4" ϕ steel bars may be applied by a torque of approximately 600 foot pounds.
4. Handling Pretensioned Deck Units: When the deck units are handled they shall be lifted by the devices provided in the tops at the ends of the units. Other types of lifting devices than those shown on the plans may be used provided they meet with the approval of the Contracting Officer
5. Prior to placing the deck, the top surfaces of the boxes shall be prepared in accordance with the specifications for a construction joint.

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
BRIDGE
SUPERSTRUCTURE CONCRETE DETAILS

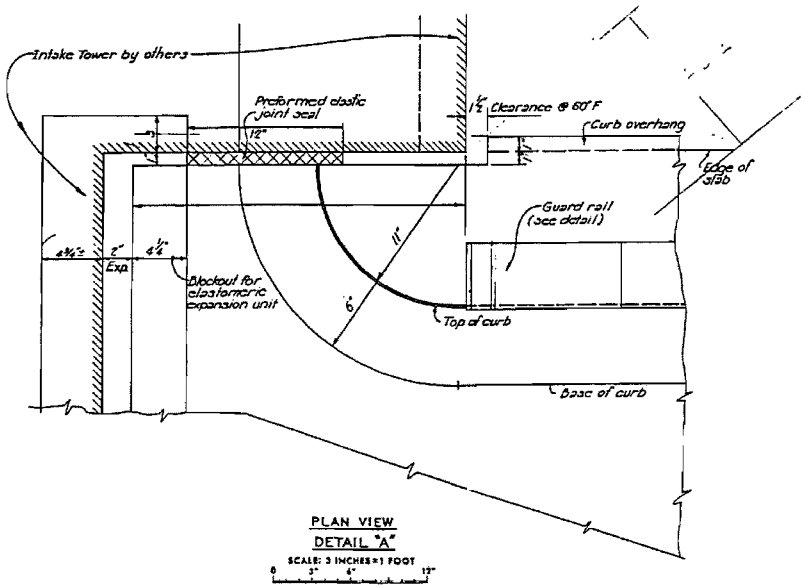
DESIGNED BY: T.A.O.
DRAWN BY: M.B.H.
CHECKED BY: H.W.H.
SUBMITTED BY: Chief Structural Section
APPROVED: Chief of Design Branch
APPROVED: R. G. Burnett, Chief Engineering Division
APPROVED: B. P. Sandegren, Lt. Col., C.E., District Engineer
DATE: NOV 1971
SCALE: AS SHOWN
SPEC. NO. DACW45 72 B 0049
DRAWING NUMBER
SHEET PC9-50E55



SLAB PLAN

SCALE: 1/2 INCH = 1 FOOT

NOTE: Shape deck transition to cause deck drainage to flow smoothly to drains in intake tower



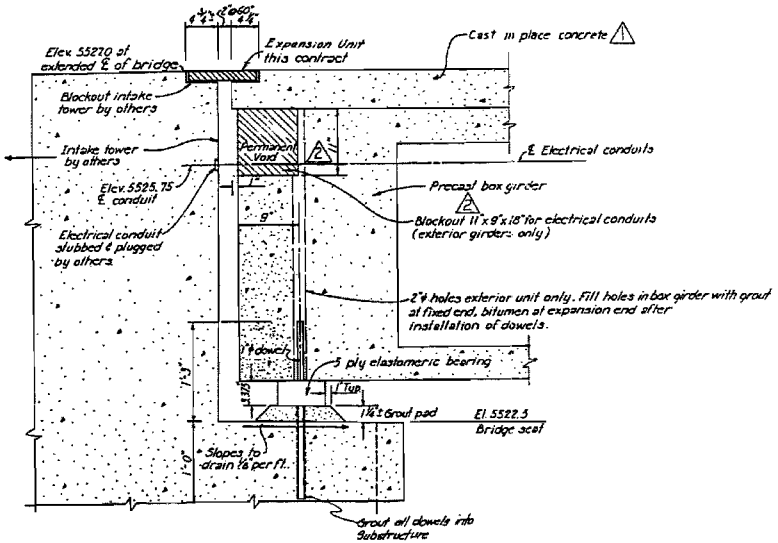
PLAN VIEW

DETAIL "A"

SCALE: 3 INCHES = 1 FOOT

NOTES:

1. Size of blockout may vary to accomodate the dimensions of the expansion connectors.
2. A galvanized steel box may be used to form permanent blockout for conduit connectors.
3. All elastomeric pads shall be laminated from five equal thickness elastomeric pads of 50 durometer hardness bonded to non-corrosive metal shims.
4. Preformed elastic joint seals shall conform to AASHO material specification M220-67. The adhesive used shall be Delastibond No. 1520 manufactured by D. S. Brown Co.
5. Expansion units shall be Transflex No. 200A or approved equal.



EXPANSION DETAILS AT INTAKE TOWER

SCALE: 1 1/2 INCHES = 1 FOOT

THIS DRAWING HAS BEEN REDUCED TO ONE-HALF THE ORIGINAL SCALE

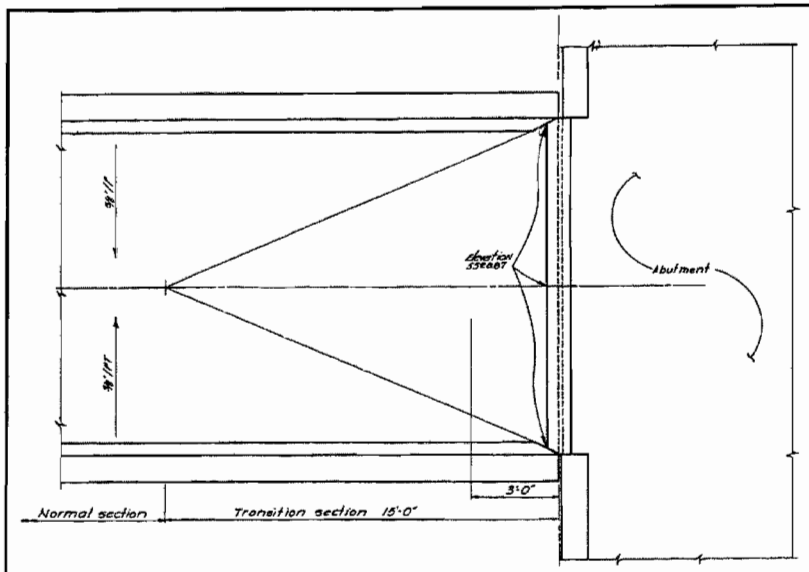
DATE	REVISIONS	BY	APPROVED
12-27-71	REVISED IN ACCORDANCE WITH AM. NO. 0002		
12-9-71	REVISED IN ACCORDANCE WITH AM. NO. 0001		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
BRIDGE
SUPERSTRUCTURE
MISC. DETAILS SHEET I

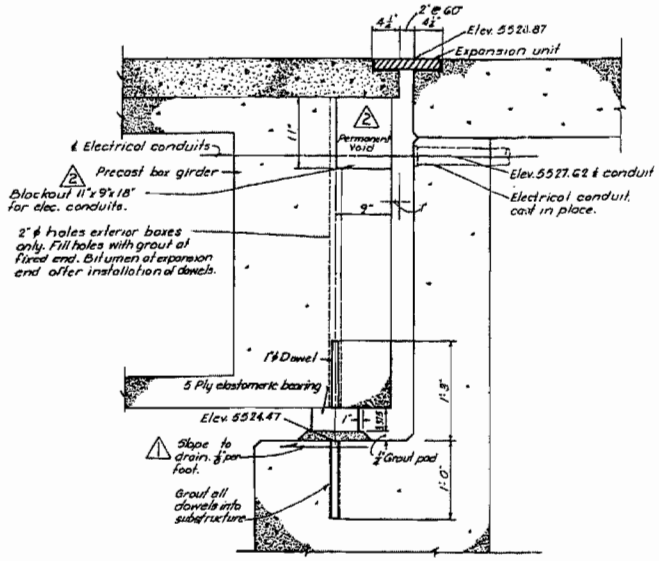
DESIGNED BY: T. A.B.
DRAWN BY: T.G.F.
CHECKED BY: H.W.H.
DESIGN STRUCT. ENGINEER
DESIGN DRAUGHTSMAN
APPROVED: R. P. Burnett
DATE: NOV. 1971
SCALE AS SHOWN
APPROVED: B. P. Lindgren
DRAWING NUMBER: PC9-50E57

THIS PLAN ACCOMPANIES CONTRACT NO. MODIFICATION NO.



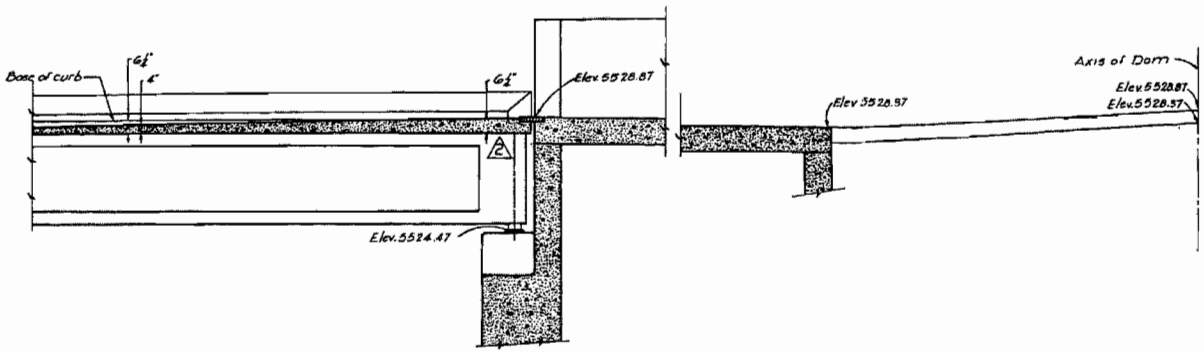
PLAN

SCALE: 1½ INCHES = 1 FOOT



BEARING & CONDUIT DETAIL AT ABUTMENT

SCALE: 1½ INCHES = 1 FOOT



SECTION AT CENTERLINE

SCALE: 1½ INCHES = 1 FOOT

THIS DRAWING HAS BEEN REDUCED TO ONE-HALF THE ORIGINAL SCALE

DATE	DESCRIPTION	MADE	APPROVED
12.27.71	REVISED IN ACCORDANCE WITH AM. NO. 0002		
12.9.71	REVISED IN ACCORDANCE WITH AM. NO. 0001		

REVISIONS

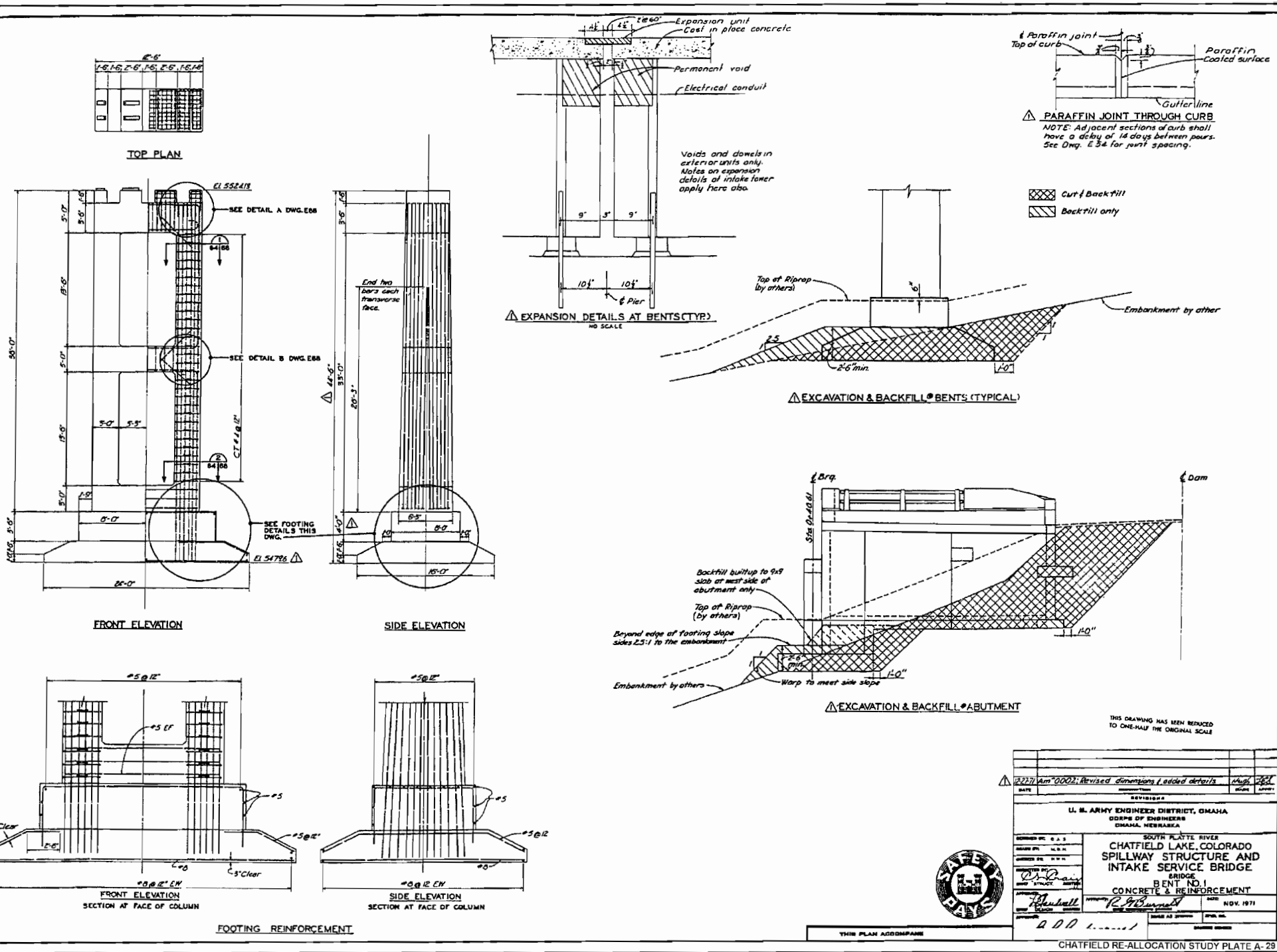
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

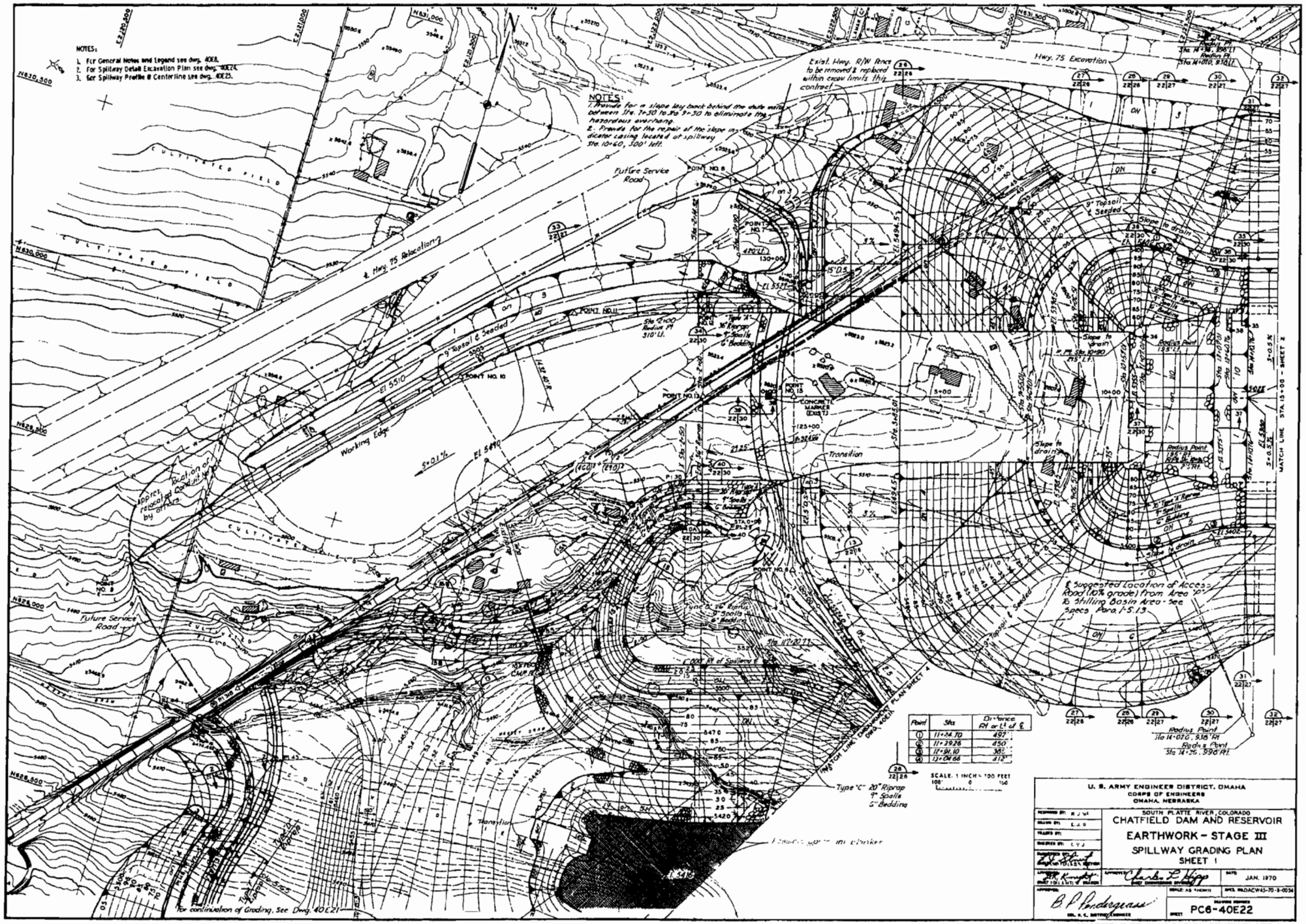
SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
BRIDGE
SUPERSTRUCTURE
MISC. DETAILS SHEET II

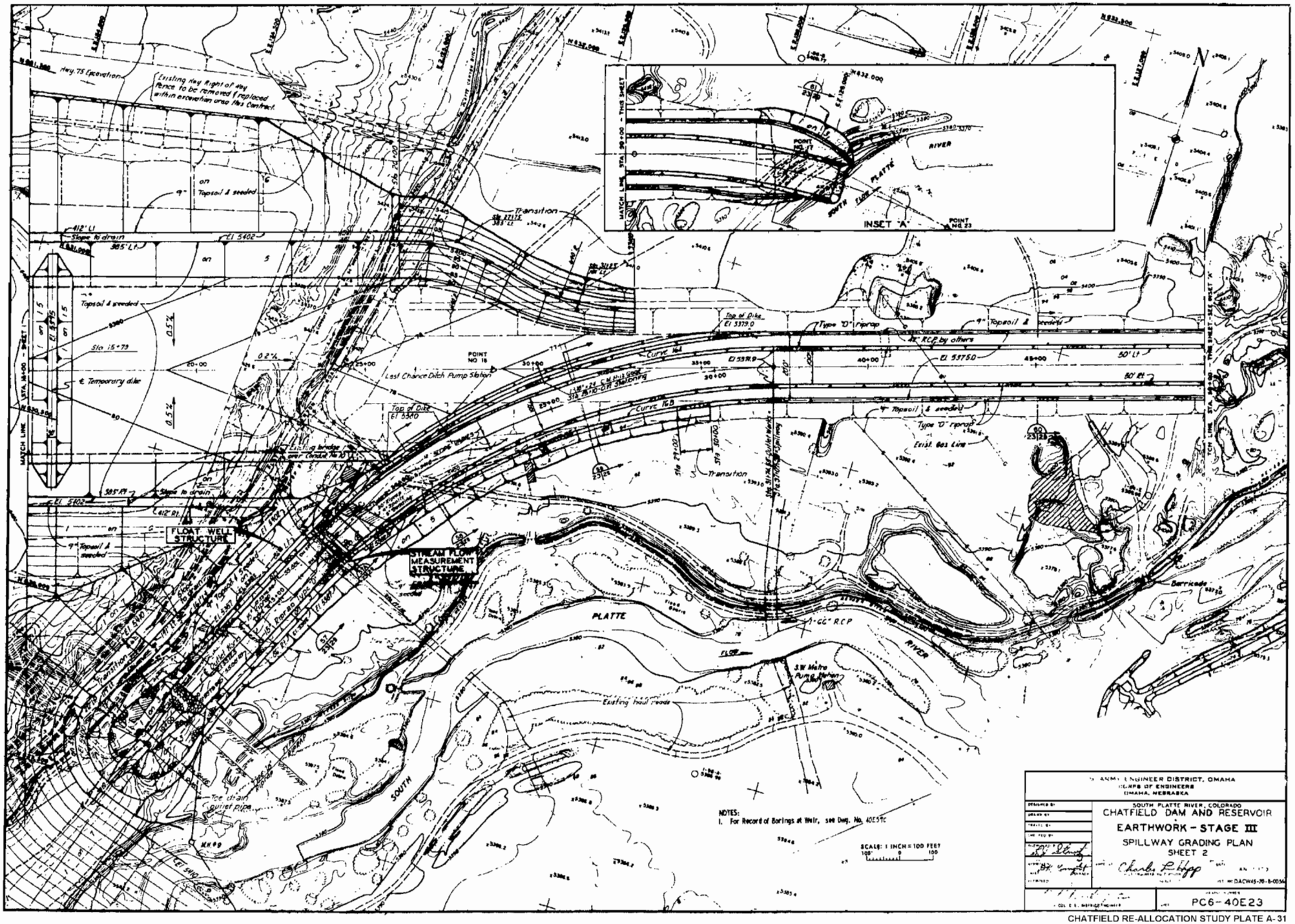
DESIGNED BY: D.J.B.
DRAWN BY: T.G.F.
CHECKED BY: K.W.K.
SUBMITTED BY: C. S. Craig, CHIEF, STRUCT. BRANCH
APPROVED: H. Neubert, CHIEF, DESIGN BRANCH
APPROVED: C. J. Burgett, CHIEF, DESIGN SECTION
APPROVED: B. L. Sandagness
DATE: NOV. 1971
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER: PCQ-50E58

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT NO.


TOP PLAN
FRONT ELEVATION
SIDE ELEVATION
SEE DETAIL A DWG. E68
SEE DETAIL B DWG. E68
SEE FOOTING DETAILS THIS DWG.
EL 55.24 ±
EL 54.79 ±
End two bars each transverse face.
Expansion unit
Cast in place concrete
Permanent void
Electrical conduit
Voids and dowels in exterior units only. Notes on expansion details at intake tower apply here also.
℄ Pier
EXPANSION DETAILS AT BENTS (TYP.)
NO SCALE
℄ Paraffin joint
Top of curb
Paraffin Coated surface
Gutter line
PARAFFIN JOINT THROUGH CURB
NOTE: Adjacent sections of curb shall have a delay of 14 days between pours. See Dwg. E34 for joint spacing.
Cut & Backfill
Backfill only
Top of Riprop (by others)
Embankment by other
2.5
2'-6" min.
1'-0"
EXCAVATION & BACKFILL @ BENTS (TYPICAL)
℄ Brg.
℄ Dam
Backfill buttup to 9x9 slab at west side of abutment only
Top of Riprop (by others)
Beyond edge of footing slope sides 2.5:1 to the embankment
Embankment by others
Warp to meet side slope
EXCAVATION & BACKFILL @ ABUTMENT
FOOTING REINFORCEMENT
FRONT ELEVATION
SECTION AT FACE OF COLUMN
SIDE ELEVATION
SECTION AT FACE OF COLUMN
2" Clear
#5 @ 12"
#5 EF
#5
#8 @ 12" EW
#8 @ 12 EW
THIS DRAWING HAS BEEN REDUCED TO ONE-HALF THE ORIGINAL SCALE
12-27-71 Am "0002": Revised dimensions & added details
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
BRIDGE
BENT NO. 1
CONCRETE & REINFORCEMENT
NOV. 1971
THIS PLAN ACCOMPANIES


NOTES:
1. For General Notes and Legend see dwg. 40E8.
2. For Spillway Detail Excavation Plan see dwg. 40E24.
3. For Spillway Profile & Centerline see dwg. 40E25.
NOTES:
1. Provide for a slope key back behind the chute walls between Sta. 7+50 to Sta. 9+50 to eliminate the hazardous overhang.
2. Provide for the repair of the slope inclinometer casing located at spillway Sta. 10+60, 300' left.
Exist. Hwy. R/W fence to be removed & replaced within excav limits this contract
Hwy. 75 Excavation
Future Service Road
Hwy. 75 Relocation
Working Edge
Transition
Type "C" 20" Riprap 9" Spalls 5" Bedding
Type "A" 36" Riprap 9" Spalls 6" Bedding
9" Topsoil & Seeded
Slope to drain
Radius Point
Suggested Location of Access Road (10% grade) from Area "D" to Stilling Basin Area - See Specs. Para. 1-5.13.
Point | Sta | Distance Rt or Lt of ℄
① 11+34.70 | 497'
② 11+39.26 | 450'
③ 12+91.10 | 395'
④ 13+04.66 | 412'
SCALE: 1 INCH = 100 FEET
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EARTHWORK - STAGE III
SPILLWAY GRADING PLAN
SHEET 1
JAN. 1970
PC6-40E22
For continuation of Grading, See Dwg. 40E21



NOTES:
1. For Record of Borings at Weir, see Dwg. No. 40E57C

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EARTHWORK - STAGE III
SPILLWAY GRADING PLAN
SHEET 2

DESIGNED BY:
DRAWN BY:
TRACED BY:
CHECKED BY:
SUBMITTED BY:
APPROVED BY:
ESTIMATED:

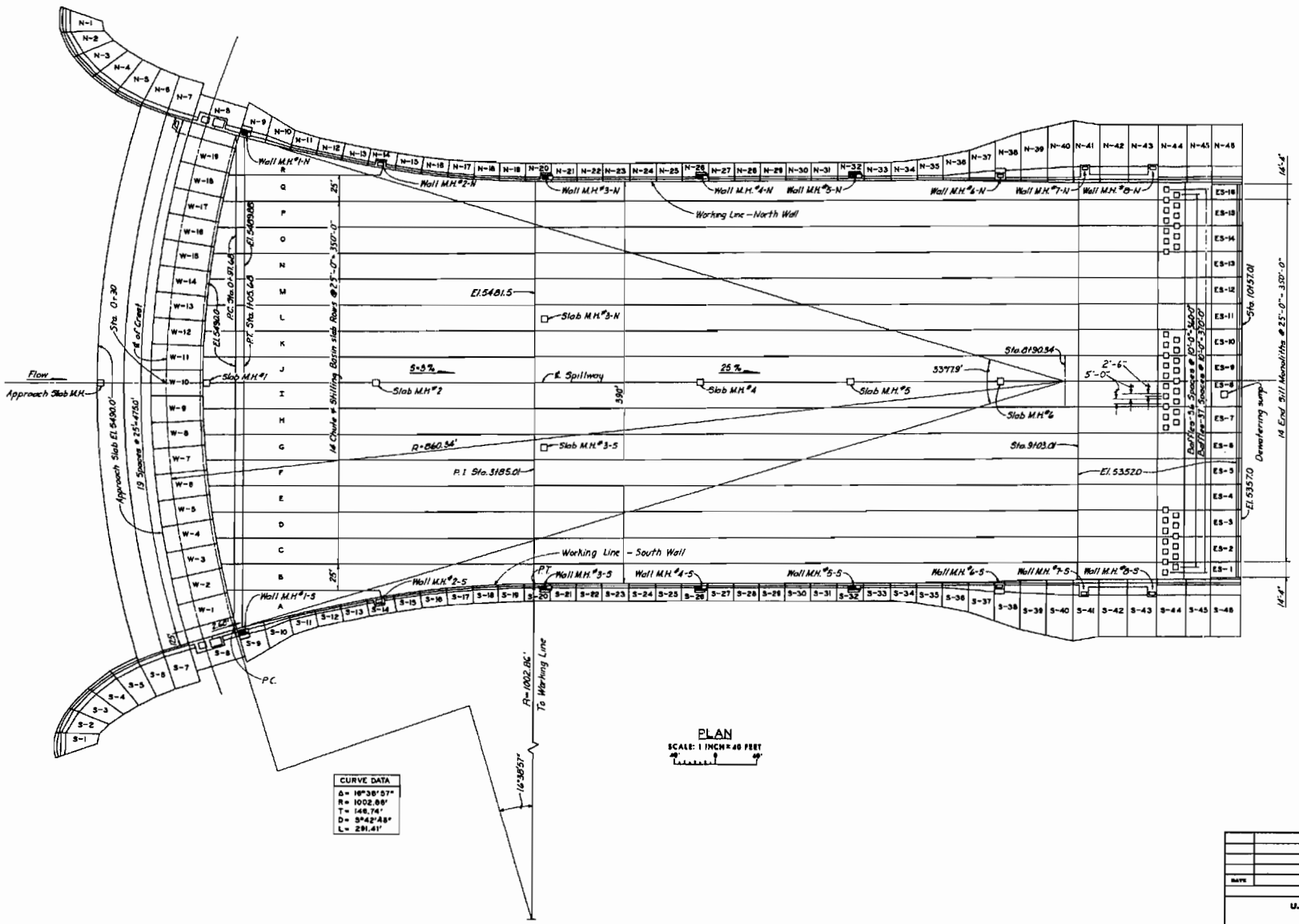
Charles P. Hipp
CHIEF, ENGINEERING DIVISION

CONTRACT NO. DACW45-70-B-0056

COL. U.S. ARMY, DISTRICT ENGINEER

PC6-40E23

CHATFIELD RE-ALLOCATION STUDY PLATE A-31



PLAN
SCALE: 1 INCH = 40 FEET

CURVE DATA
Δ = 16°38'57"
R = 1002.86'
T = 148.74'
D = 5°42'48"
L = 291.41'

DATE	DESCRIPTION	MADE	APPD
	REVISIONS		

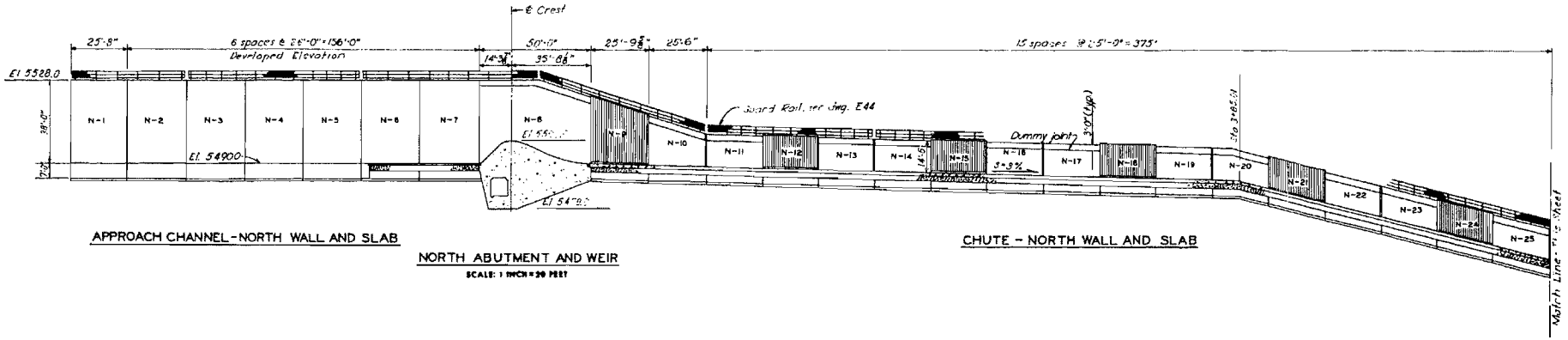
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
SPILLWAY
STRUCTURAL ARRANGEMENT
PLAN

DESIGNED BY: E. K. J.
DRAWN BY: R. D. C.
CHECKED BY: H. K. H.
SUBMITTED BY: Chief, Struct. Br.
APPROVED: Chief, Design Branch
APPROVED: Chief, Engineering Division
DATE: NOV. 1971
SCALE AS SHOWN
APPROVED: Col., C. E., District Engineer
DRAWING NUMBER: PC 9-50E13
SHEET

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

SAFETY PAYS

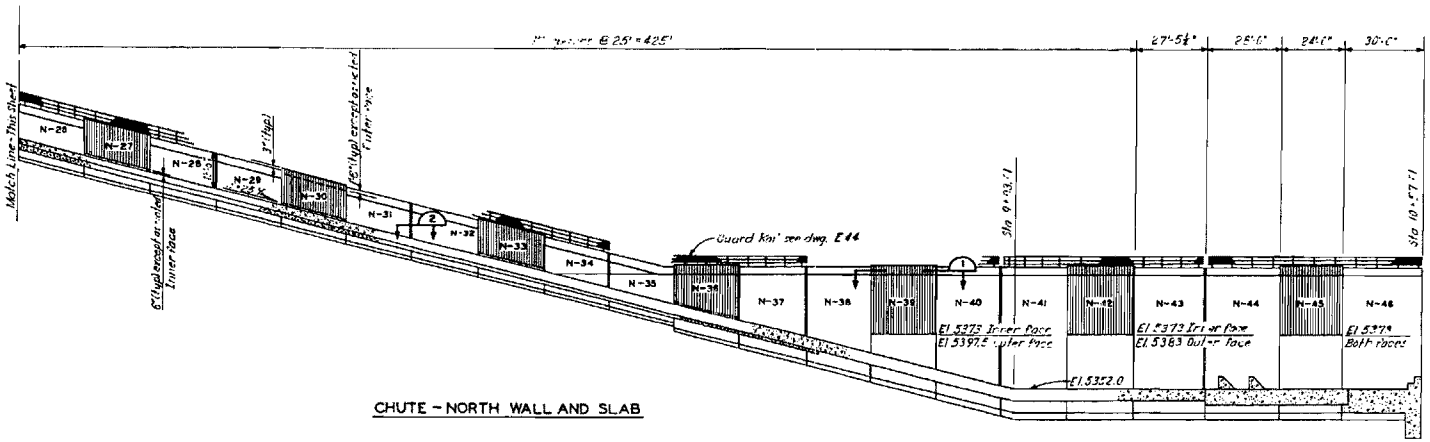


APPROACH CHANNEL - NORTH WALL AND SLAB

NORTH ABUTMENT AND WEIR

SCALE: 1 INCH = 20 FEET

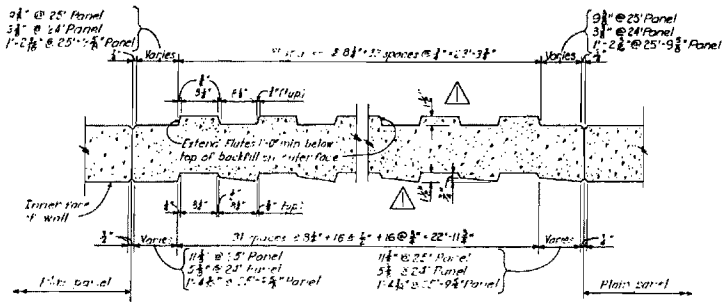
CHUTE - NORTH WALL AND SLAB



CHUTE - NORTH WALL AND SLAB

STILLING BASIN - NORTH WALL AND SLAB

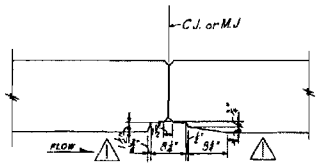
SCALE: 1 INCH = 20 FEET



SECTION 1

TYPICAL SECTION THROUGH FLUTES

SCALE: 1 INCH = 1 FOOT



SECTION 2

TYPICAL SECTION THROUGH VERTICAL JOINTS BETWEEN FLUTED PANELS

SCALE: 1 INCH = 1 FOOT.

Note:
North Wall Shown.
South Wall Opp. Hand.

DATE	DESCRIPTION	DRAWN	APPRVD
12-9-71	REVISED IN ACCORDANCE WITH AN. NO. 0001		

REVISIONS

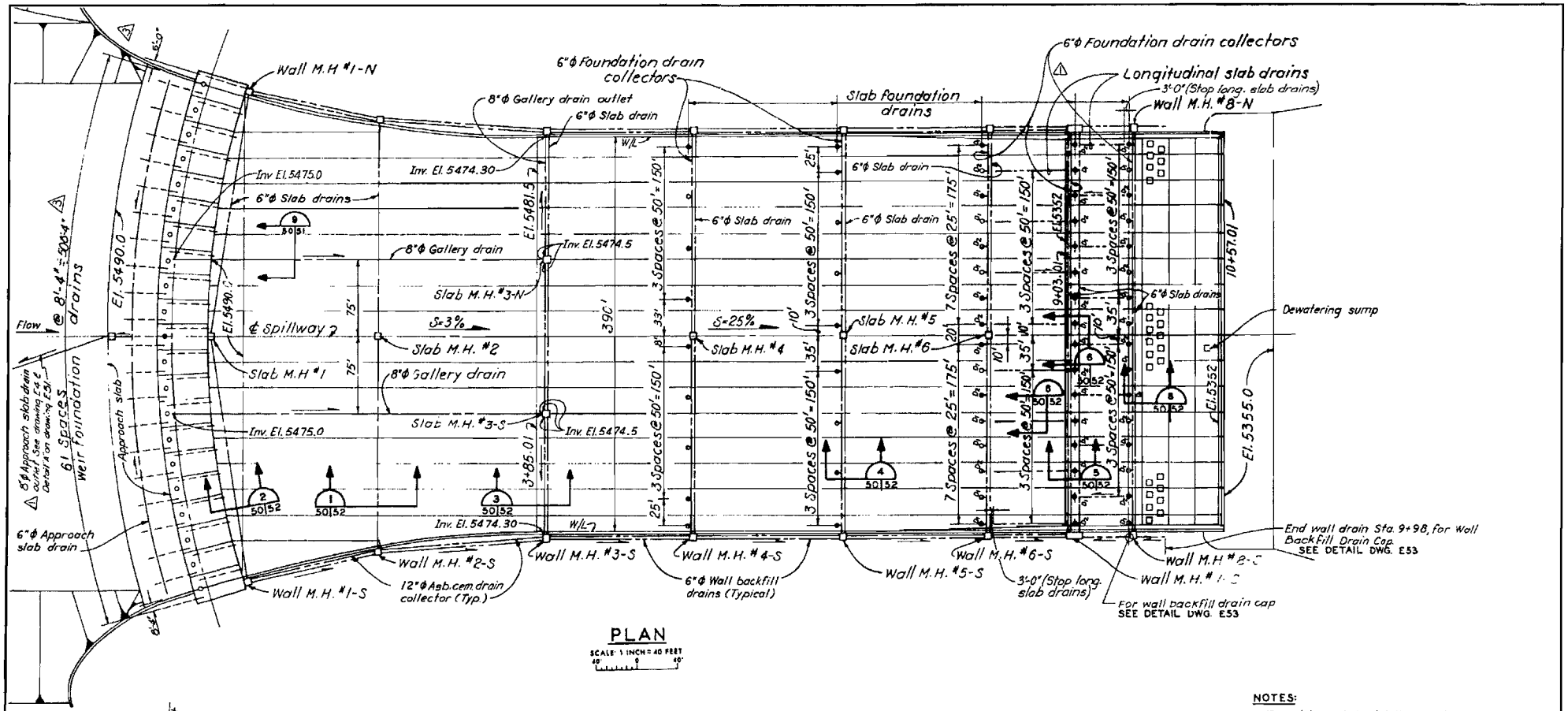
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
SPILLWAY
STRUCTURAL ARRANGEMENT & ARCHITECTURAL TREATMENT - PROFILE AND SECTIONS

DESIGNED BY: E. W. J.
DRAWN BY: W. L. M.
CHECKED BY: H. W. M.
STRUCT. ENGR.
CHIEF, DESIGN BRANCH
CHIEF, ENGINEERING DIVISION
DATE: NOV. 1971
SCALE: AS SHOWN
SHEET: PC9-50E14

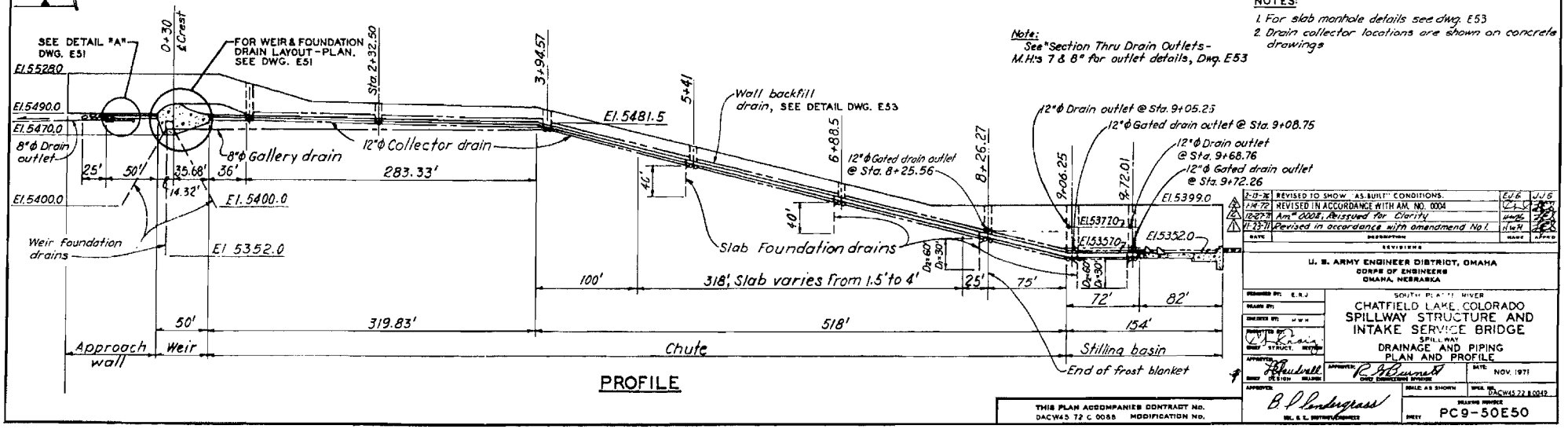
THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

SAFETY PAYS



PLAN

SCALE: 1 INCH = 40 FEET



PROFILE

NOTES:

1. For slab manhole details see dwg. E53
2. Drain collector locations are shown on concrete drawings

Note: See "Section Thru Drain Outlets - M.H.'s 7 & 8" for outlet details, Dwg. E53

DATE	DESCRIPTION	DRAWN	APPRD
2-13-76	REVISED TO SHOW "AS BUILT" CONDITIONS.	EJG	JJG
1-14-72	REVISED IN ACCORDANCE WITH AM. NO. 0004		
12-27-71	Am" 0008, Reissued for Clarity		
11-23-71	Revised in accordance with amendment No 1		

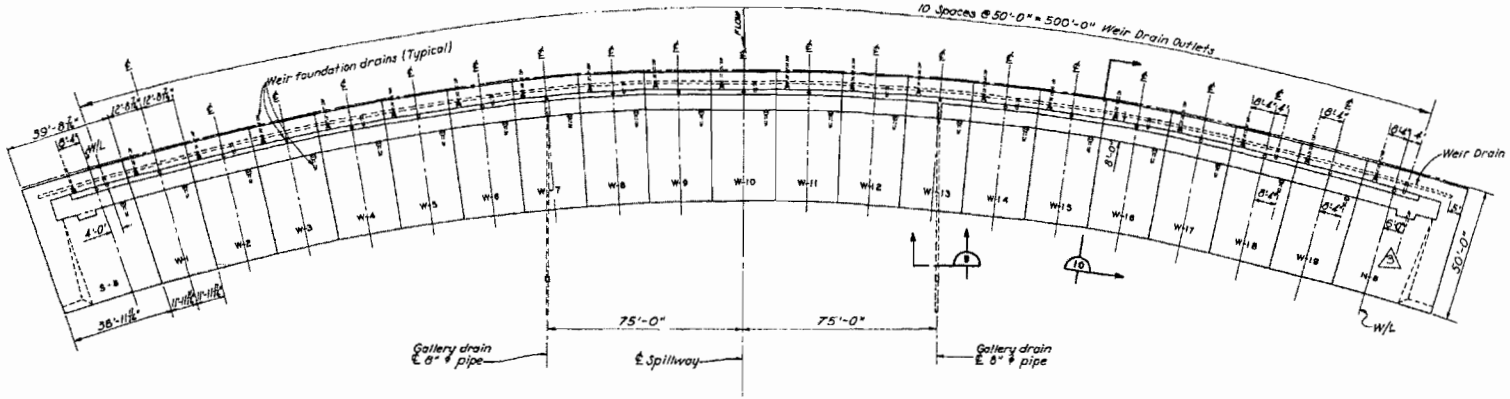
REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
SPILLWAY
DRAINAGE AND PIPING
PLAN AND PROFILE

DESIGNED BY: E.R.J.
DRAWN BY:
ENGINEER BY: H.W.H.
DESIGN STRUCT. REVIEW
APPROVED: DIVISION ENGINEER
APPROVED: CHIEF, ENGINEERING DIVISION
APPROVED: B. P. Sandergrass, DIV. & S.E. DISTRICT ENGINEER
DATE: NOV. 1971
SCALE: AS SHOWN
SPEC. NO. DACW45 72 B 0049
DRAWING NUMBER
SHEET PC9-50E50

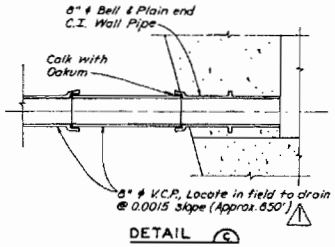
THIS PLAN ACCOMPANIES CONTRACT No. DACW45 72 C 0085 MODIFICATION No.



PLAN

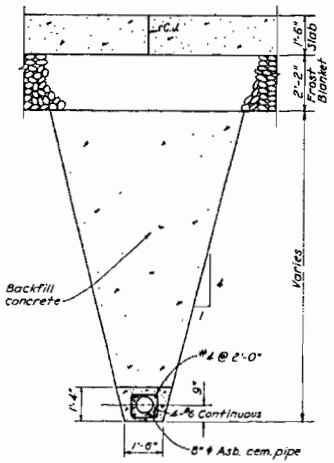
WEIR & FOUNDATION DRAIN LAYOUT

SCALE: 1 INCH = 20 FEET



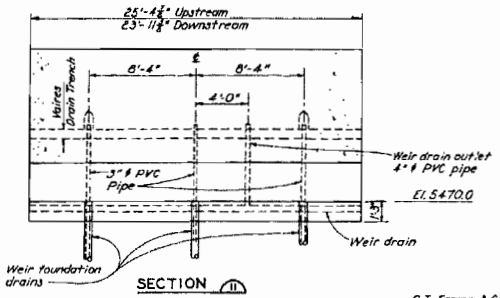
DETAIL (C)

NO SCALE

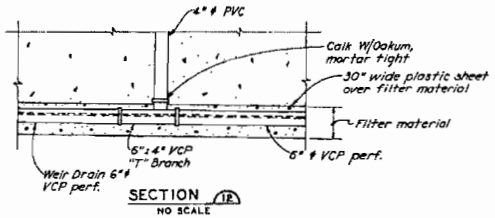


SECTION (50|51)

SCALE: 1/4 INCH = 1 FOOT

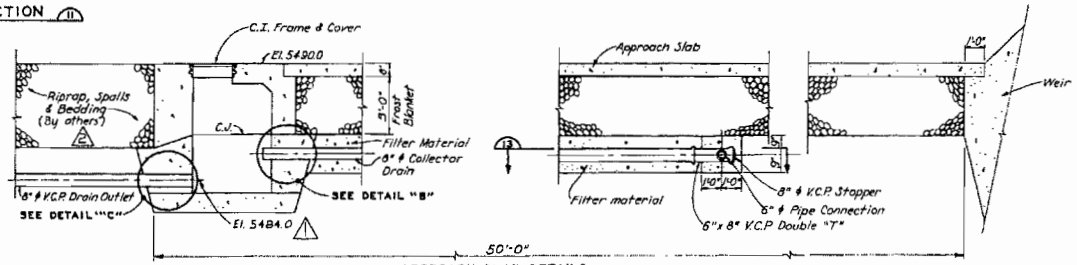


SECTION (11)



SECTION (12)

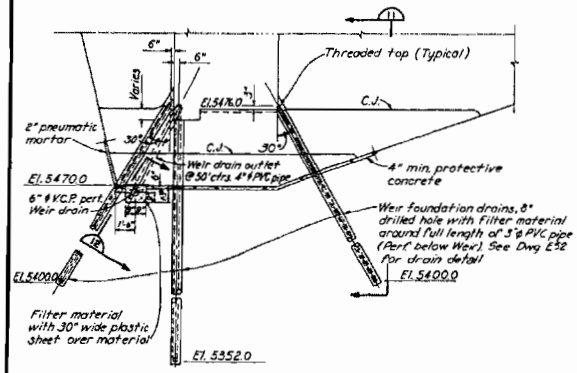
NO SCALE



APPROACH SLAB DETAILS

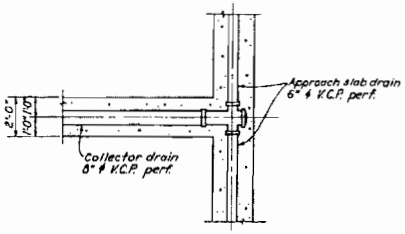
DETAIL (A)

SCALE: 1/4 INCH = 1 FOOT



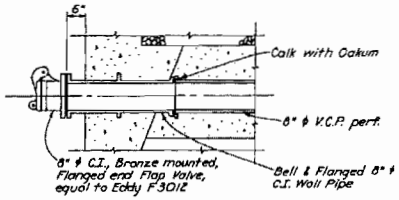
SECTION (10)

SCALE: 1/4 INCH = 1 FOOT



SECTION (13)

SCALE: 1/4 INCH = 1 FOOT



DETAIL (B)

NO SCALE

DATE	DESCRIPTION	MADE	APPR'D
2-23-76	REVISED TO SHOW "AS-BUILT" CONDITIONS.	EJG	JJG
1-14-72	REVISED IN ACCORDANCE WITH AM. NO. 0004		
12-27-71	REVISED IN ACCORDANCE WITH AM. NO. 0002		
12-9-71	REVISED IN ACCORDANCE WITH AM. NO. 0001		

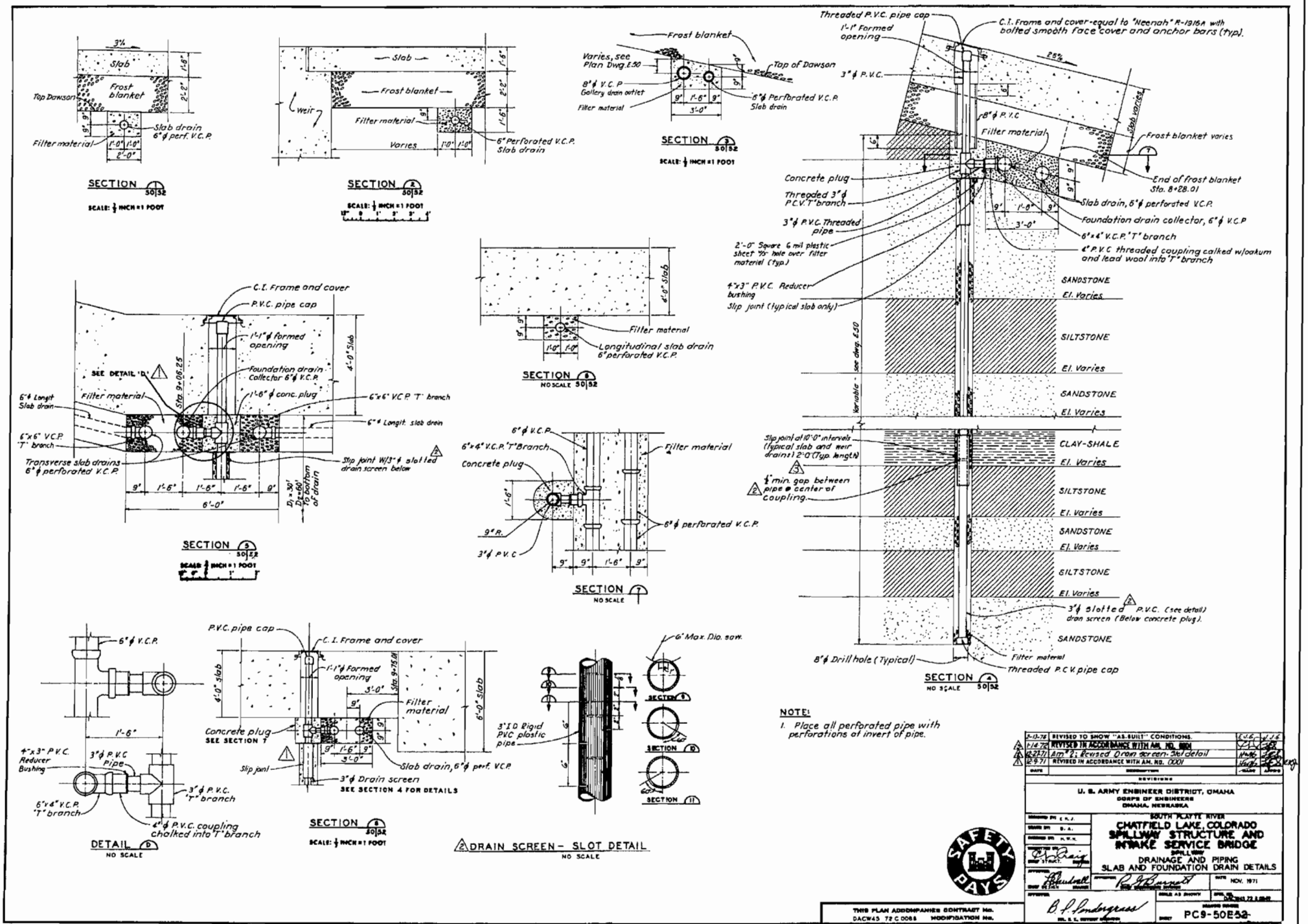
REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND
INTAKE SERVICE BRIDGE
SPILLWAY
DRAINAGE AND PIPING
APPROACH SLAB & WEIR DRAIN DETAILS

DESIGNED BY: E. R. J.
DRAWN BY: L. M. G.
REVIEWED BY: W. W. H.
SUBMITTED BY: STRUCT. DESIGN
APPROVED: CHIEF, DESIGN BRANCH
APPROVED: CHIEF, ENGINEERING DIVISION
APPROVED: CHIEF, CONSTRUCTION DIVISION
DATE: NOV. 1971
SCALE: AS SHOWN
SPEC. NO. DACW45 72 B 0049
SHEET: PC9-50E51

THIS PLAN ACCOMPANIES CONTRACT NO. DACW45 72 C 0088 MODIFICATION NO.



NOTE:

1. Place all perforated pipe with perforations at invert of pipe.

5-10-78	REVISED TO SHOW "AS-BUILT" CONDITIONS.	E.J.C.	J.J.C.
2 1-14-76	REVISED IN ACCORDANCE WITH AM. NO. 0004		
1 12-27-71	Am. 2; Revised Drain screen-Slot detail		
1 12-9-71	REVISED IN ACCORDANCE WITH AM. NO. 0001		
DATE	DESCRIPTION	DRAWN	APPRD

REVISIONS

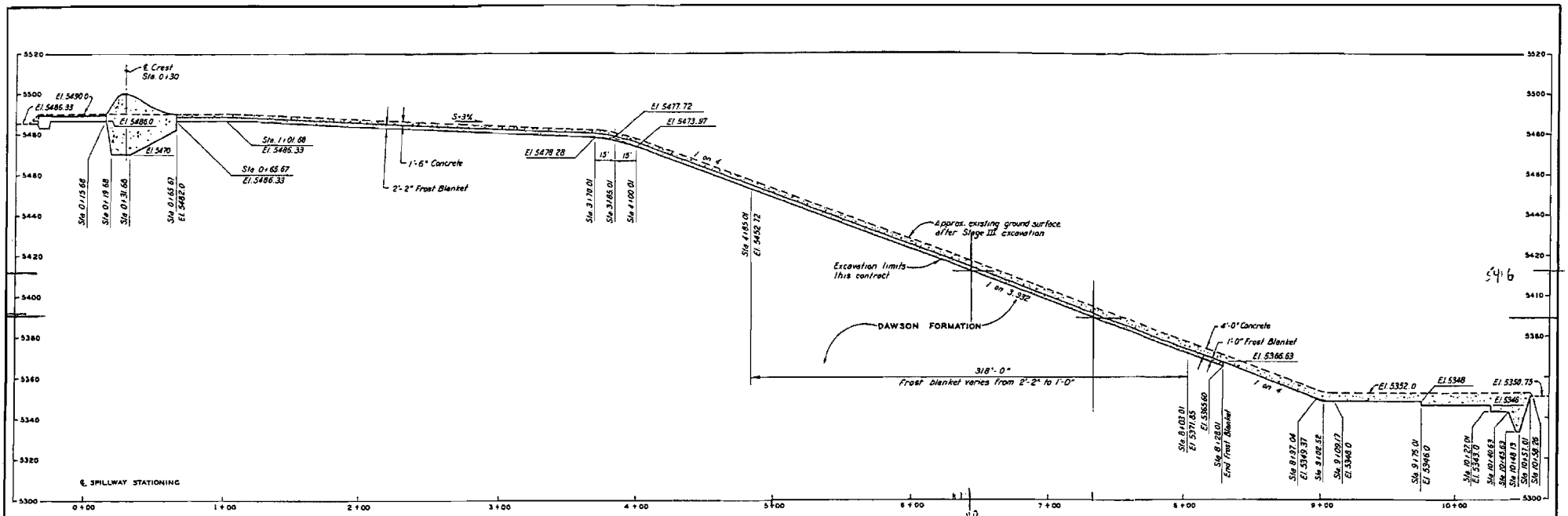
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
SPILLWAY
DRAINAGE AND PIPING
SLAB AND FOUNDATION DRAIN DETAILS

DESIGNED BY: E.K.J.
DRAWN BY: D.A.
CHECKED BY: K.W.K.
DESIGN STRUCT.
DESIGN CIVIL
DRAWN
APPROVED
CHIEF, ENGINEERING DIVISION
DATE: NOV. 1971
SCALE AS SHOWN
SPEC. NO. DACW45-72-C-0088
APPROVED
DIR. S. & E. DIVISION
SHEET PC9-50E52
DRAWING NUMBER

THIS PLAN ACCOMPANIES CONTRACT No. DACW45 72 C 0088 MODIFICATION No.

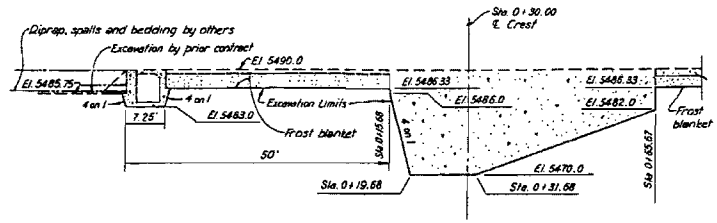
SAFETY PAYS



PROFILE ALONG SPILLWAY ℄

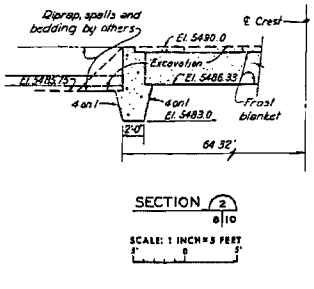
ELEVATION IN FEET ABOVE M.S.L.

SCALE: VERT. 1 INCH = 20 FEET
HORIZ. 1 INCH = 30 FEET



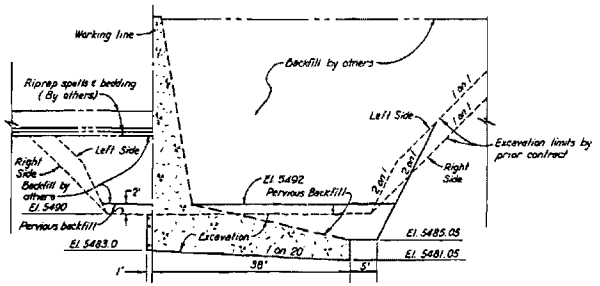
SECTION 1 / 8|10

SCALE: 1 INCH = 10 FEET



SECTION 2 / 8|10

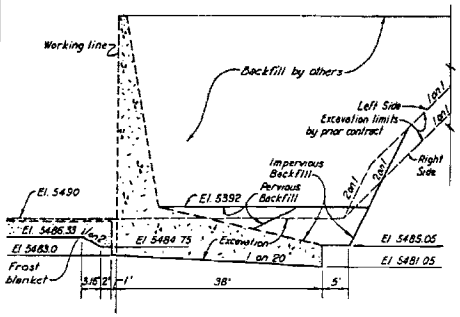
SCALE: 1 INCH = 5 FEET



SECTION 3 / 8|10

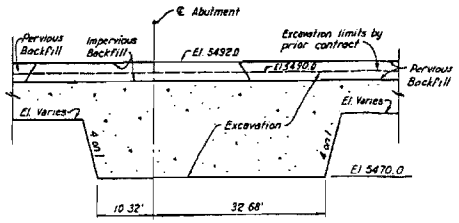
SCALE: 1 INCH = 10 FEET

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.



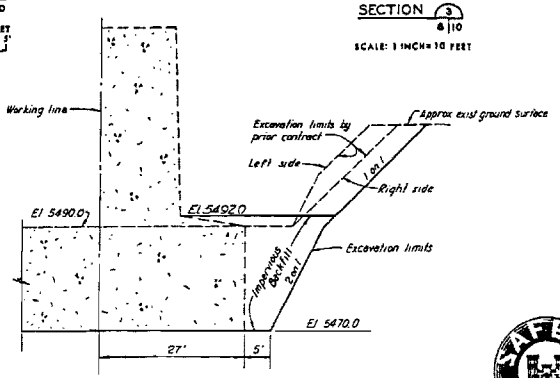
SECTION 4 / 8|10

SCALE: 1 INCH = 10 FEET



SECTION 5 / 8|10

SCALE: 1 INCH = 10 FEET



SECTION 6 / 8|10

SCALE: 1 INCH = 10 FEET

DATE	DESCRIPTION	MADE	APPRD
2-13-74	REVISED TO SHOW "AS-BUILT" CONDITIONS.	R.J.G.	J.J.G.

REVISIONS

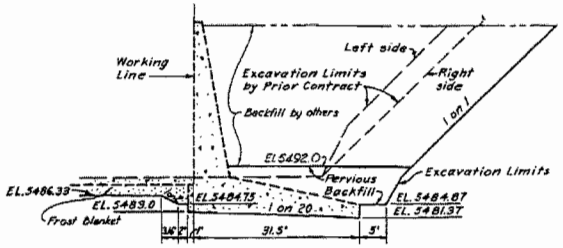
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
SPILLWAY
PROFILE AND STRUCTURAL EXCAVATION SECTIONS

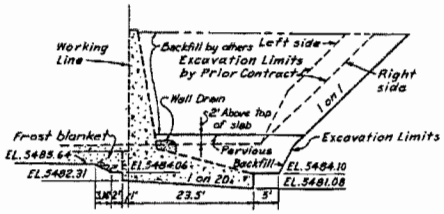
DESIGNED BY: R. J. H.
DRAWN BY: E. H. B.
CHECKED BY: C. W. J.
DATE: NOV. 1971
SCALE: AS SHOWN
SPEC. NO. DACW45-72-B-0049

THIS PLAN ACCOMPANIES CONTRACT No. DACW45-72-C-0044 MODIFICATION No.

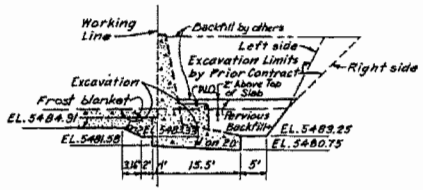
SHEET PC9-50E10



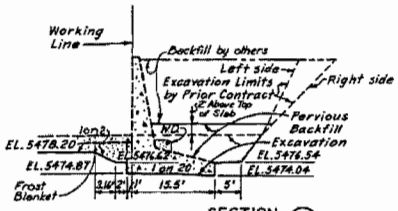
SECTION 7 / 8|11



SECTION 8 / 8|11

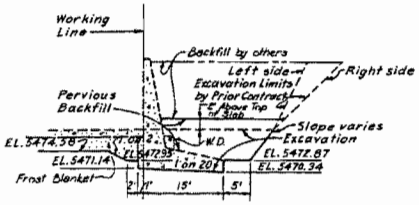


SECTION 9 / 8|11



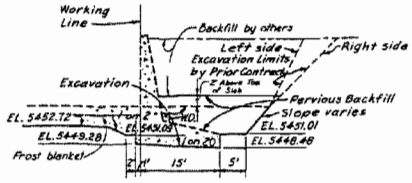
SECTION 10 / 8|11

STA. 3+72.57



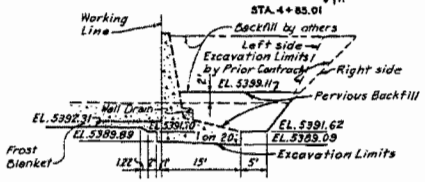
SECTION 11 / 8|11

STA. 3+97.57



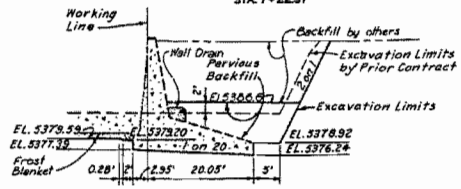
SECTION 12 / 8|11

STA. 4+85.01



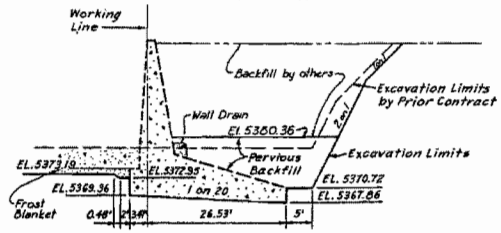
SECTION 13 / 8|11

STA. 7+22.57



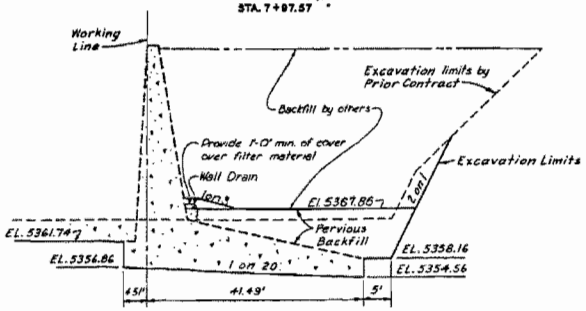
SECTION 14 / 8|11

STA. 7+72.57



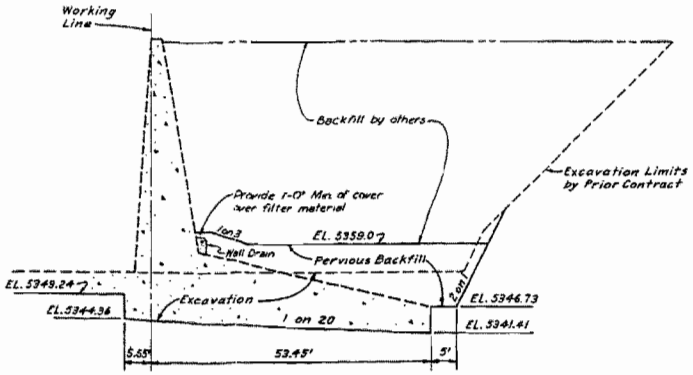
SECTION 15 / 8|11

STA. 7+97.57



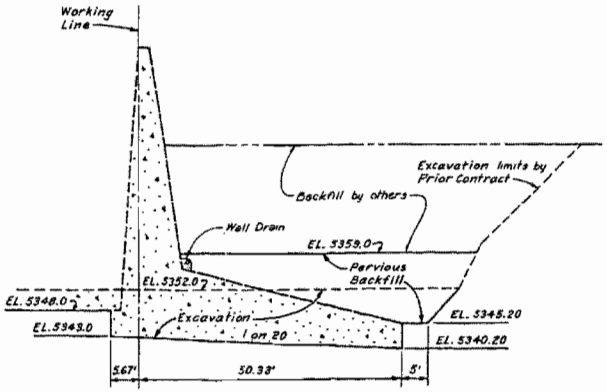
SECTION 16 / 8|11

STA. 8+47.57



SECTION 17 / 8|11

STA. 8+97.57



SECTION 18 / 8|11

Note:
1. For details of wall drains see dwgs. nos. 50E50 & 50E53.

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION	DRAWN	APPRD
7-13-76	REVISED TO SHOW "AS-BUILT" CONDITIONS.	C.J.G.	J.J.A.
4-9-72	REVISED IN ACCORDANCE WITH AM. NO. 0001	[illegible]	[illegible]

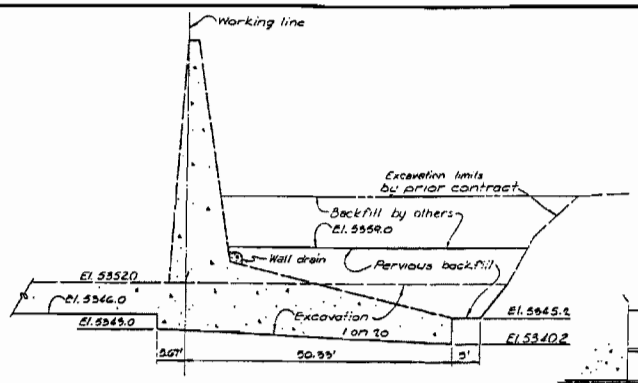
REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

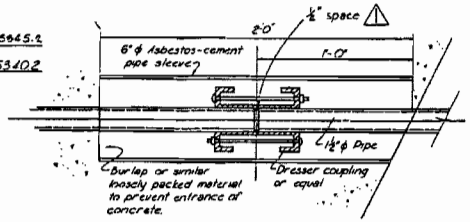
SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
SPILLWAY
STRUCTURAL EXCAVATION SECTIONS
SHEET 1

DESIGNED BY: R.J.W.
DRAWN BY:
TRACED BY:
CHECKED BY: C.V.J.
PROJECT ENGINEER FOR DESIGN: [illegible]
PROJECT ENGINEER FOR CONSTRUCTION: [illegible]
APPROVED: R. P. Burnett, CHIEF, ENGINEERING DIVISION
DATE: NOV. 1971
APPROVED: R. P. Sandegren, CHIEF, CONSTRUCTION DIVISION
SCALE:
CONTRACT NO. DACW45 72 C 0049
DRAWING NUMBER: PC9-50E11

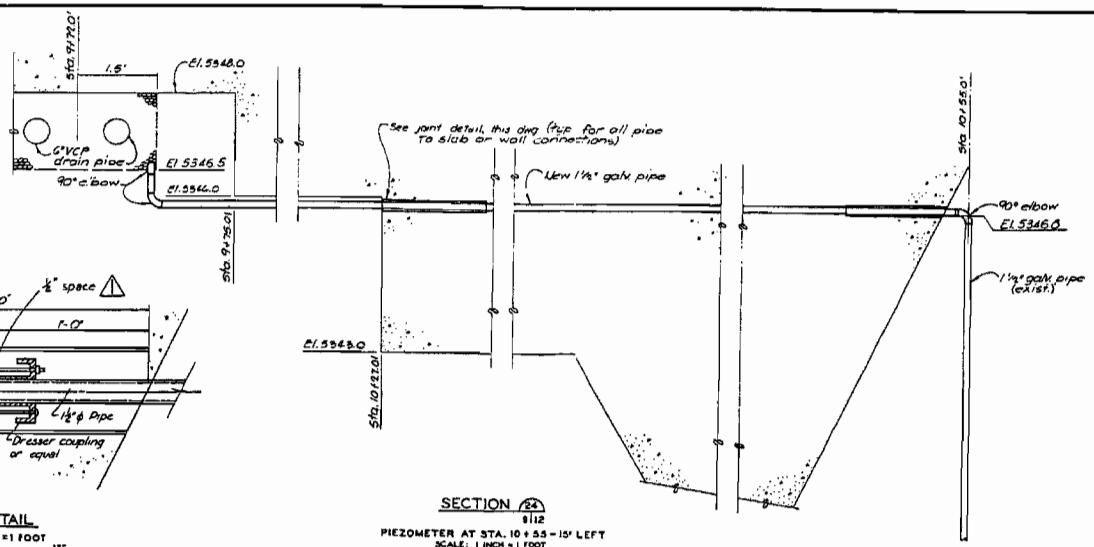
THIS PLAN ACCOMPANIES CONTRACT NO. DACW45 72 C 0049 MODIFICATION NO.



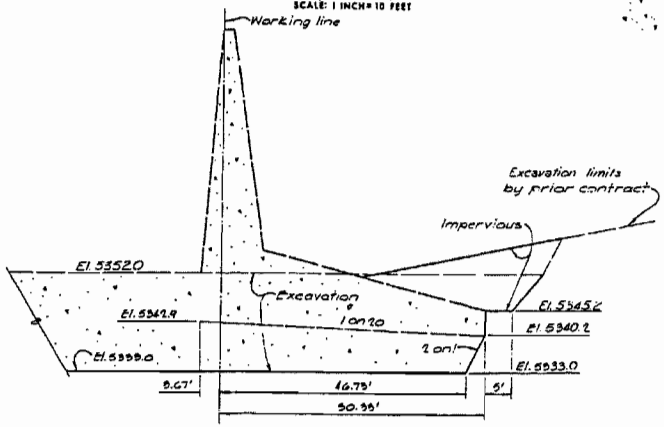
SECTION 19
9|12
SCALE: 1 INCH = 10 FEET



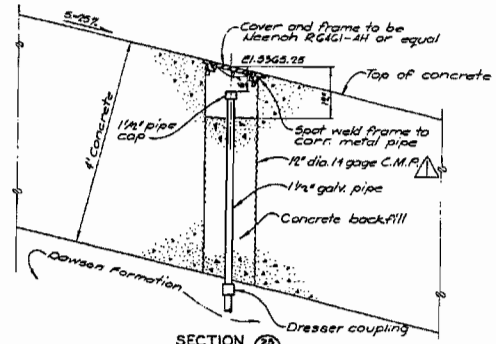
JOINT DETAIL
SCALE: 3 INCHES = 1 FOOT



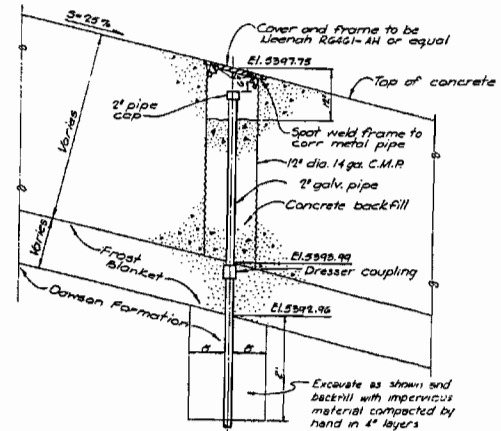
SECTION 24
9|12
PIEZOMETER AT STA. 10+55 – 15' LEFT
SCALE: 1 INCH = 1 FOOT



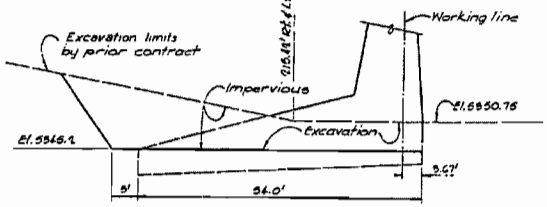
SECTION 20
9|12
SCALE: 1 INCH = 10 FEET



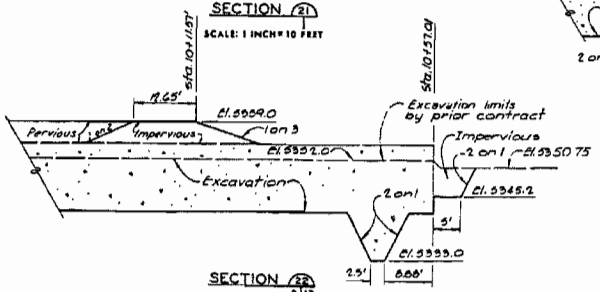
SECTION 25
9|12
PIEZOMETER AT STA. 8+50 – 15' RIGHT
SCALE: 1 INCH = 1 FOOT



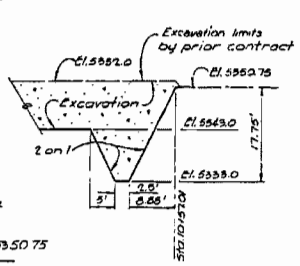
SECTION 26
9|12
PIEZOMETER AT STA. 7+20 – 5' RIGHT
SCALE: 1 INCH = 1 FOOT



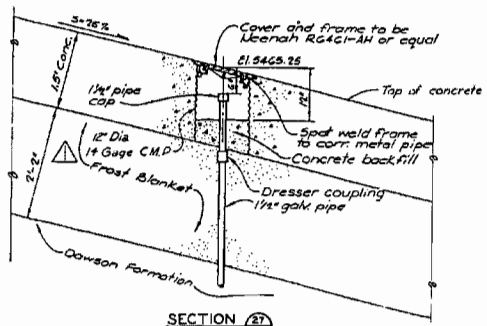
SECTION 21
SCALE: 1 INCH = 10 FEET



SECTION 22
9|12
SCALE: 1 INCH = 10 FEET



SECTION 23
9|12
SCALE: 1 INCH = 10 FEET



SECTION 27
9|12
PIEZOMETER AT STA. 4+50 – 15' LEFT
SCALE: 1 INCH = 1 FOOT

SCALE: 1 INCH = 1 FOOT

SCALE: 1 INCH = 10 FEET

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION	MADE	APPRD
2-13-76	REVISED TO SHOW "AS-BUILT" CONDITIONS.	EJG	JJG
12-27-71	REVISED TO [illegible] WITH AM. NO. 0002.	[illegible]	[illegible]

REVISIONS

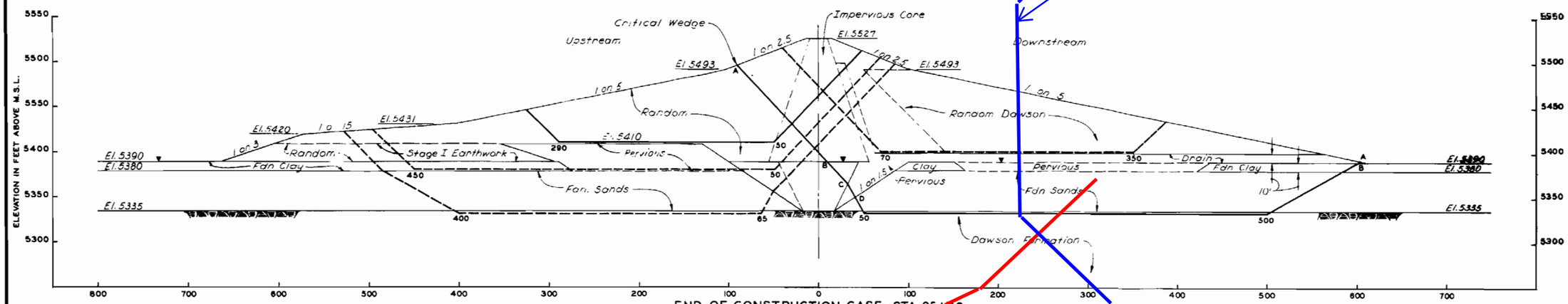
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
SPILLWAY STRUCTURE AND INTAKE SERVICE BRIDGE
SPILLWAY
STRUCTURAL EXCAVATIONS AND SECTIONS
SHEET 2

DESIGNED BY: A.J.M.
DRAWN BY: D.E.H.
CHECKED BY: C.V.J.
SUBMITTED BY: [illegible] CHIEF, STN. & S.S. SECTION
APPROVED: [illegible] CHIEF, STN. & MTL'S. BRANCH
APPROVED: [illegible] CHIEF, ENGINEERING DIVISION
DATE: NOV. 1971
APPROVED: [illegible] COL., C.E., DISTRICT ENGINEER
SCALE: AS SHOWN
SPEC. NO. DACW45-72-B-0049
SHEET PC9-50E12

THIS PLAN ACCOMPANIES CONTRACT No. DACW45-72-C-0066 MODIFICATION No.

SAFETY PAYS



END OF CONSTRUCTION CASE - STA. 95+00

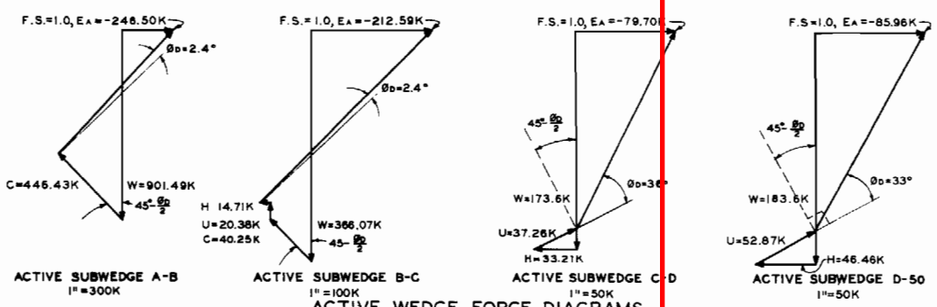
SCALE: 1 INCH = 50 FEET

FACTORS OF SAFETY UPSTREAM				
ELEVATION OF FAILURE PLANE	ACTIVE WEDGE DISTANCE FROM EMB. ℄	PASSIVE WEDGE DISTANCE FROM EMB. ℄	FACTORS OF SAFTY	
			NORMAL	EARTHQUAKE
5410	50	290	2.62	1.82*
5390	50	450	2.71	1.84
5335	65	400	2.64	1.75

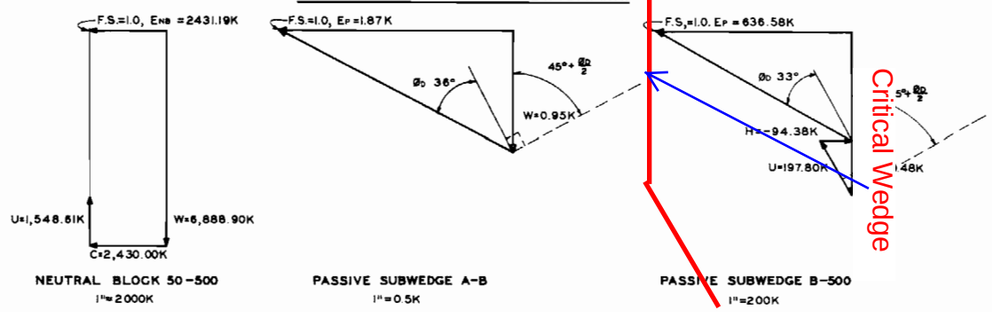
* CRITICAL

FACTORS OF SAFETY DOWNSTREAM				
ELEVATION OF FAILURE PLANE	ACTIVE WEDGE DISTANCE FROM EMB. ℄	PASSIVE WEDGE DISTANCE FROM EMB. ℄	FACTORS OF SAFETY	
			NORMAL	EARTHQUAKE
5400	70	350	2.67	1.90
5335	50	500	2.53	1.62*

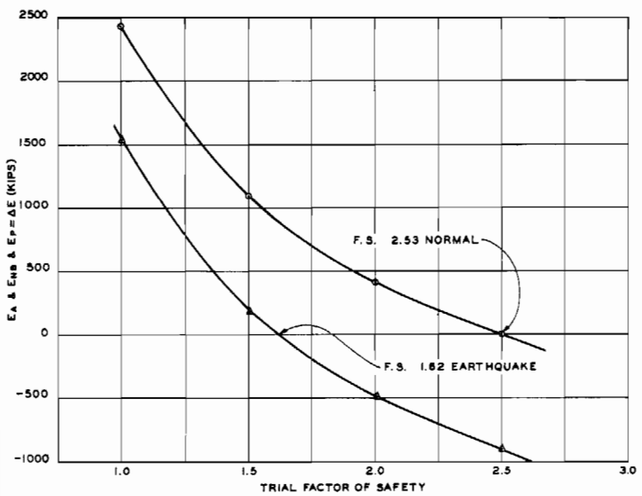
*CRITICAL



ACTIVE WEDGE FORCE DIAGRAMS



NEUTRAL BLOCK AND PASSIVE WEDGE FORCE DIAGRAMS



NOTES:

1. Net Hydrostatic Force is perpendicular to vertical sides of subwedge or block.
2. Seismic coefficient = 0.1g
3. Wedges and force diagrams shown are for a trial F.S. = 1.0. Failure surfaces and force diagrams are slightly different for other trial F.S.'s due to Øb and c.
4. All stability computations performed by R.C.A. 301 Computer - Program No. 41-R3-1302C.
5. Critical wedge shown by solid line.

NOMENCLATURE

W = WEIGHT OF WEDGE OR BLOCK
H = NET HYDROSTATIC FORCE
U = UPLIFT ON FAILURE PLANE
C = (COHESION ÷ TRIAL F.S.) × LENGTH OF PLANE
Øb = ARCTAN TAN Ø ÷ TRIAL F.S.
F.S. = SHEAR STRENGTH ASSUMED / SHEAR STRENGTH NEEDED FOR EQUILIBRIUM

ADOPTED SHEAR STRENGTH						
MATERIAL	IN PLACE WEIGHT	SAT'D. WEIGHT	SUBM. WEIGHT	UNCONSOLIDATED UNDRAINED - "Q"		
	#/FT³	#/FT	#/FT	TAN Ø	Ø	COH-T/SF
EMBANKMENT						
Impervious	120.0	126.0	63.6	0.042	2.4	1.50
Random	120.0	126.0	63.6	0.042	2.4	1.50
Pervious	130.0	136.0	73.6	0.73	36.1	0.0
Dawson FN.	120.0	126.0	63.6	0.00	0	2.0
FOUNDATION						
Clay		118.0	55.6	0.00	0	0.28
Sand		125.0	62.6	0.65	33.0	0.00
Dawson	130.0	135.0	72.6	0.00	0	2.70

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: L. G. S.
DRAWN BY: R. G. S.
TRACED BY:
CHECKED BY: L. G. S.
SUBMITTED BY: [illegible] CHIEF, FDN. & MTLS. BRANCH
APPROVED: [illegible] CHIEF, ENGINEERING DIVISION

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EMBANKMENT AND EXCAVATION
STABILITY ANALYSIS
END OF CONSTRUCTION CASE
STA. 95+00

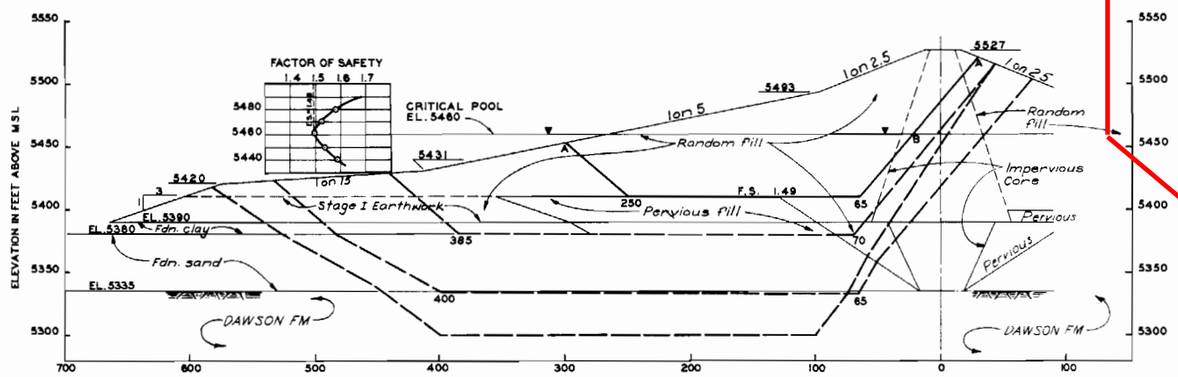
DATE: DEC. 1968
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET

APPROVED: [illegible] COL. C. E. DISTRICT ENGINEER

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

SAFETY PAYS

CHATFIELD RE-ALLOCATION STUDY PLATE B-1



PARTIAL POOL STA. 95+00

SCALE: 1 INCH = 50 FEET
50' 0 50'

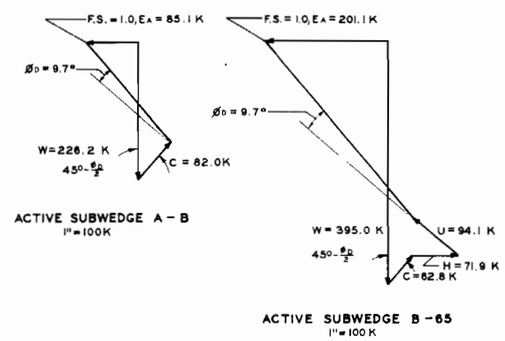
Critical Wedge

ELEV. OF FAILURE PLANE	ACTIVE WEDGE	PASSIVE WEDGE	FACTORS OF SAFETY POOL ELEVATIONS 5420 NORM.	5420 E.Q.	5431 NORM.	5431 E.Q.	5440 NORM.	5440 E.Q.	5450 NORM.	5450 E.Q.	5460 NORM.	5460 E.Q.	5470 NORM.	5470 E.Q.	5480 NORM.	5480 E.Q.
5410	65	250					1.59	1.12	1.53	1.08	1.49*	1.04	1.52	1.04	1.58	1.05
5380	70	385					1.96	1.31	1.95	1.28	1.94	1.25				
5335	65	400					2.09	1.31	2.12	1.30	2.18	1.30	*R* STRENGTH			
5300	100	400	1.78	1.12	1.74	1.08	1.76	1.08	1.79	1.07	1.84	1.07	*S* STRENGTH			

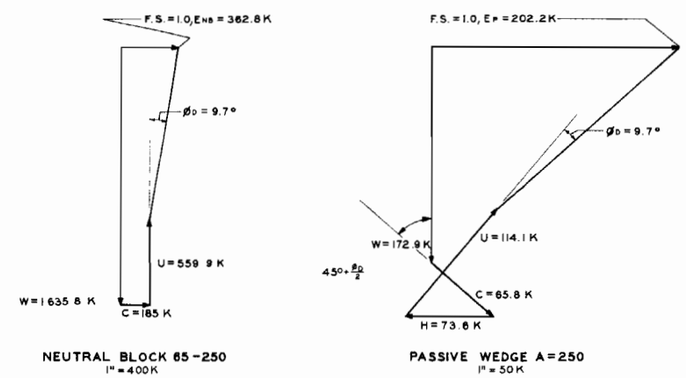
* Critical

ADOPTED DESIGN VALUES FOR EMBANKMENT STABILITY									
MATERIAL	IN PLACE WEIGHT #/FT.³	SAT'D WEIGHT #/FT.³	SUBM. WEIGHT #/FT.³	CONSOLIDATED UNDRAINED *R* TAN Ø	Ø	COH-TSF	CONSOLIDATED DRAINED *S* TAN Ø	Ø	COH-TSF
EMBANKMENT									
Impervious	120.0	126.0	63.6	0.17	9.7	0.50	0.45	24.3	0.0
Random	120.0	126.0	63.6	0.17	9.7	0.50	0.45	24.3	0.0
Pervious	130.0	136.0	73.6	0.73	36.1	0.00	0.73	36.1	0.0
Dawson Fm.	120.0	126.0	63.6	0.20	11.3	0.40	0.41	22.3	0.0
FOUNDATION									
Clay		118.0	55.6	0.15	8.5	0.30	0.38	20.8	0.0
Sand		125.0	62.6	0.65	33.0	.00	0.65	33.0	0.0
Dawson	130.0	135.0	72.6	0.35	19.3	0.40	0.27*	15.0*	0.0*

* Used in lieu of "S" strength

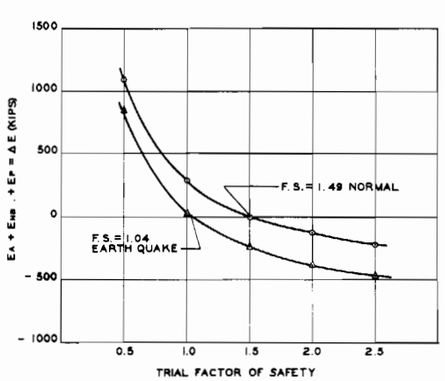


ACTIVE WEDGE FORCE DIAGRAMS



NEUTRAL BLOCK AND PASSIVE WEDGE FORCE DIAGRAMS

NOTES:
For notes see plate B-73



THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

NOMENCLATURE

W = WEIGHT OF WEDGE OR BLOCK
H = NET HYDROSTATIC FORCE
U = UPLIFT ON FAILURE PLANE
C = (COHESION + TRIAL F.S.) × LENGTH OF PLANE
ϕ_D = ARCTAN TAN ϕ ÷ TRIAL F.S.
F.S. = SHEAR STRENGTH ASSUMED / SHEAR STRENGTH NEEDED FOR EQUILIBRIUM

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

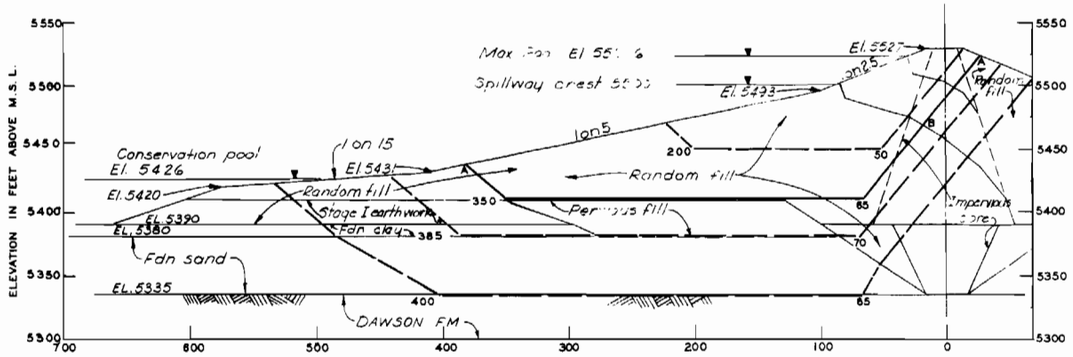
DESIGNED BY: L.G.S
DRAWN BY: J.M.F
TRACED BY:
CHECKED BY: L.G.S
SUBMITTED BY: Chief, Div. Hydraulic Div.
APPROVED: Chief, Div. H. & M. Br.

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EMBANKMENT AND EXCAVATION
STABILITY ANALYSIS
PARTIAL POOL CASE
STA. 95 + 00

APPROVED: Chief, Engineering Division
DATE: DEC 1968
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET

APPROVED: COL. G. E., DISTRICT ENGINEER

CHATFIELD RE-ALLOCATION STUDY PLATE B-2

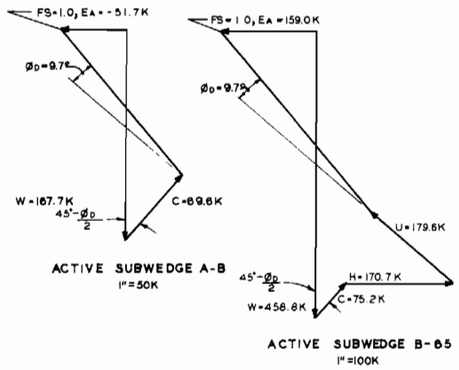


SUDDEN DRAWDOWN STA. 95+00

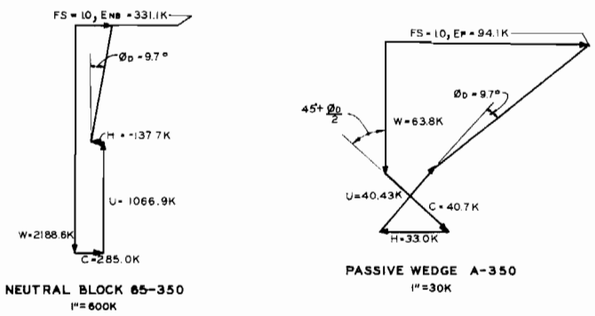
SCALE: 1 INCH = 50 FEET

FACTORS OF SAFETY						
EL. OF FAILURE PLANE	ACTIVE WEDGE	PASSIVE WEDGE	FACTORS OF SAFETY			
			CREST		MAX.	
			NORMAL	E.Q.	NORMAL	E.Q.
5450	50	200	1.57	1.12	1.50	–
5410	65	350	1.33*	0.92	1.23	–
5380	70	385	1.56	1.05	1.47	–
5335	65	400	1.78	1.14	1.68	–
Req'd F.S.			1.20	1.0	1.0	–

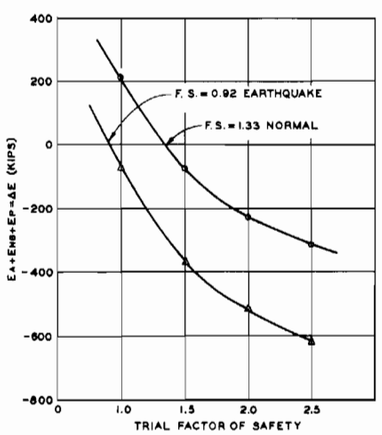
*Critical



ACTIVE WEDGE FORCE DIAGRAMS



NEUTRAL BLOCK AND PASSIVE WEDGE FORCE DIAGRAMS



ADOPTED SHEAR STRENGTHS						
MATERIAL	IN PLACE WEIGHT	SAT'D WEIGHT	SUBM WEIGHT	CONSOLIDATED UNDRAINED-"R"		
	#/FT³	#/FT.³	#/FT.³	TAN Ø	Ø	COHES. T/FT²
EMBANKMENT						
Impervious	120.0	126.0	63.6	0.17	9.7	0.50
Random	120.0	126.0	63.6	0.17	9.7	0.50
Pervious	130.0	136.0	73.6	0.73	36.1	0.00
Dawson FM.	120.0	126.0	63.6	0.20	11.3	0.40
FOUNDATION						
Clay		118.0	55.6	0.15	8.5	0.30
Sand		125.0	62.6	0.65	33.0	0.0
Dawson	130.0	135.0	72.6	0.35	19.3	0.40

NOMENCLATURE

W = WEIGHT OF WEDGE OR BLOCK
H = NET HYDROSTATIC FORCE
U = UPLIFT ON FAILURE PLANE
C = (COHESION ÷ TRIAL F.S.) × LENGTH OF PLANE
ϕ_D = ARCTAN TAN Ø ÷ TRIAL F.S.
F.S. = SHEAR STRENGTH ASSUMED / SHEAR STRENGTH NEEDED FOR EQUILIBRIUM

NOTE:
1. For notes, see plate B-73

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

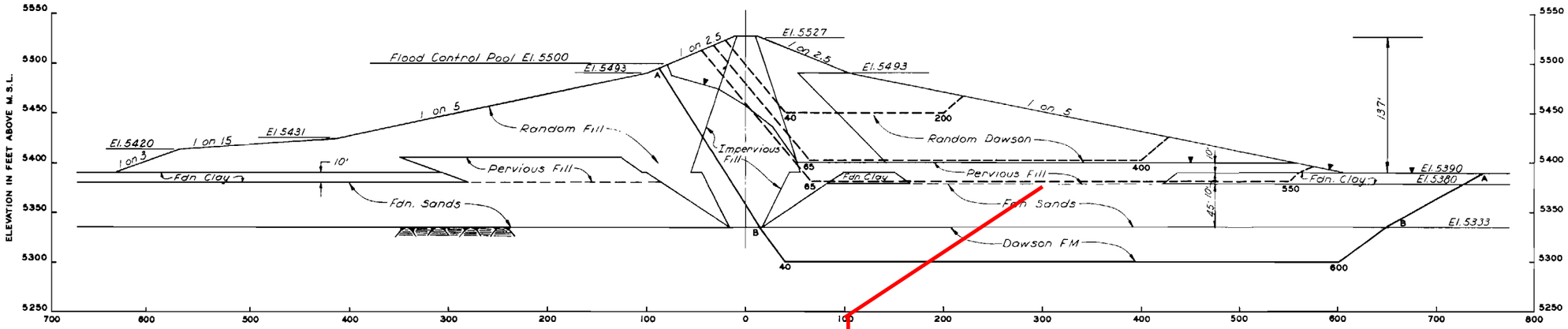
SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EMBANKMENT AND EXCAVATION

STABILITY ANALYSIS
SUDDEN DRAWDOWN CASE
STA. 95+00

DESIGNED BY: L. G. S.
DRAWN BY: R. G. H.
TRACED BY:
CHECKED BY: L. G. S.
SUBMITTED BY: CHIEF, ENG. FOUNDATION SECTION
APPROVED: CHIEF, FDN. & MTLS. BRANCH
APPROVED: CHIEF, ENGINEERING DIVISION
DATE: DEC. 1968
SCALE: AS SHOWN
SPEC. NO.
APPROVED: COL. S. E. DISTRICT ENGINEER
DRAWING NUMBER
SHEET

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

CHATFIELD RE-ALLOCATION STUDY PLATE B-3

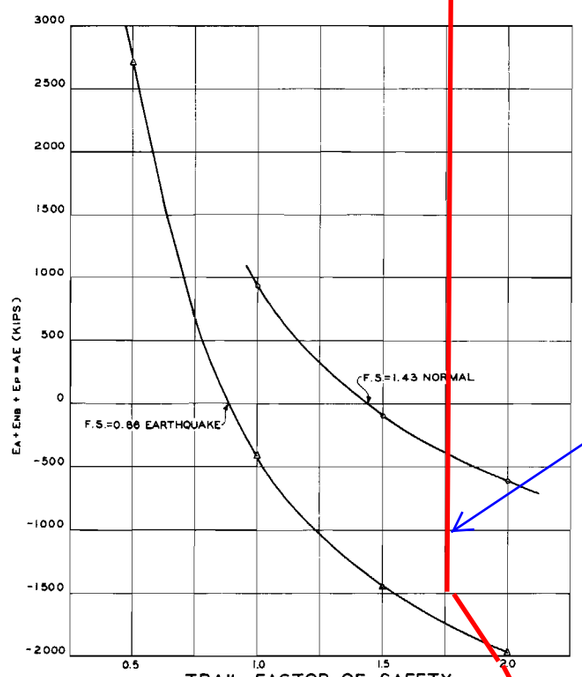


STEADY SEEPAGE STA. 95+00

SCALE: 1 INCH = 50 FEET

EL. OF FAILURE PLANE	ACTIVE WEDGE	PASSIVE WEDGE	FACTORS OF SAFETY					
			R. STRENGTH		S. STRENGTH		AVERAGE	
			NORMAL	E.Q.	NORMAL	E.Q.	NORMAL	E.Q.
5450	40 65	200 315	1.82	1.32	1.84	1.34	1.83	1.83
5400	65 85	400 565	1.63	1.13	2.13	1.48	1.88	1.31
5380	65	550	2.49	1.68	2.73	1.84	2.61	1.76
5300	40	600	1.98	1.26	*1.43	0.86		

* Critical



TRAIL FACTOR OF SAFETY

APOPTED SHEAR STRENGTHS									
MATERIAL	IN PLACE WEIGHT	SAT'D. WEIGHT	SUBM. WEIGHT	CONSOLIDATED UNDRAINED - "R"			CONSOLIDATED DRAINED - "S"		
	#/FT.³	#/FT.³	#/FT.³	TAN ϕ	ϕ	COH-T/SF	TAN ϕ	ϕ	COH-T/SF
EMBANKMENT									
Impervious	120.0	126.0	63.6	0.17	9.7	0.50	0.45	24.3	0.0
Random	.0	126.0	63.6	0.17	9.7	0.50	0.45	24.3	0.0
Pervious	.0	136.0	73.6	0.73	36.1	0.00	0.73	36.1	0.0
Dawson FM.	.0	126.0	63.6	0.20	11.3	0.40	0.41	22.3	0.0
FOUNDATION									
Clay		118.0	55.6	0.15	8.5	0.30	0.38	20.8	0.0
Sand		125.0	62.6	0.65	33.0	00	0.65	33.0	0.0
Dawson	.0	135.0	72.6	0.35	19.3	0.40	0.27*	15.0*	0.0*

* Used in lie 's" strength

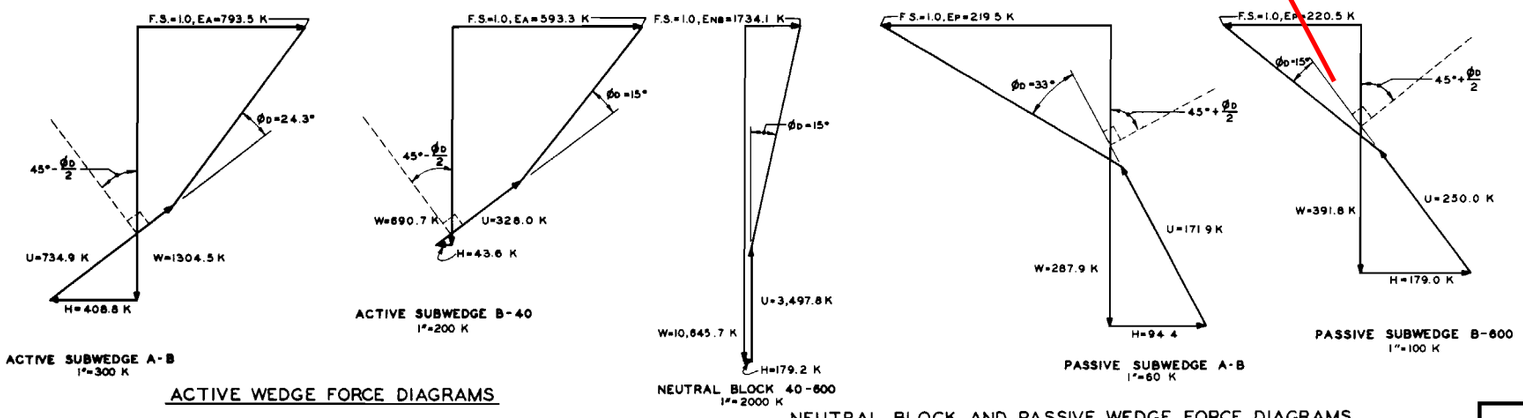
Critical Wedge

NOMENCLATURE

W = WEIGHT OF WEDGE OR BLOCK
H = NET HYDROSTATIC FORCE
U = UPLIFT ON FAILURE PLANE
C = (COHESION + TRIAL F.S.) × LENGTH OF PLANE
ϕ_D = ARCTAN TAN ϕ ÷ TRIAL F.S.
F.S. = SHEAR STRENGTH ASSUMED / SHEAR STRENGTH NEEDED FOR EQUILIBRIUM

NOTE:
1. For notes, see plate B-73

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.



ACTIVE WEDGE FORCE DIAGRAMS

NEUTRAL BLOCK AND PASSIVE WEDGE FORCE DIAGRAMS

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

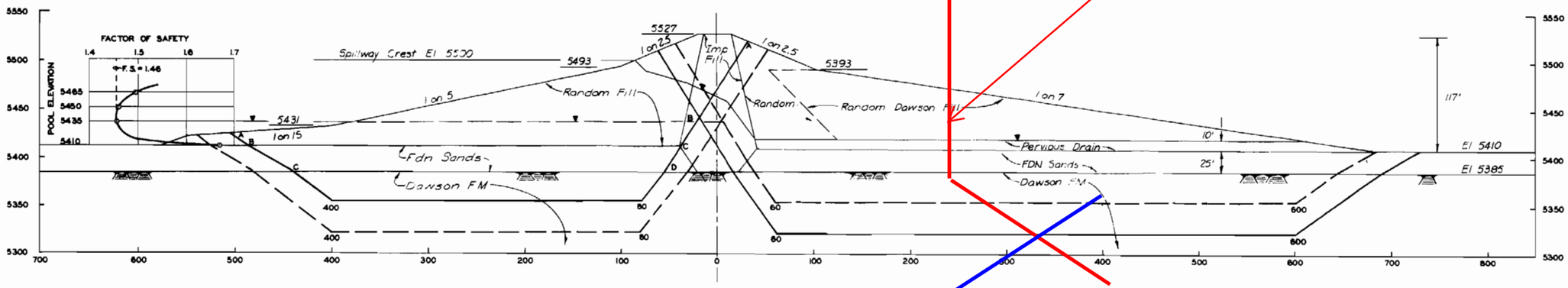
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: L. G. S.
DRAWN BY: R. N.
TRACED BY:
CHECKED BY: L. G. S.
SUBMITTED BY: [illegible] SEC.
APPROVED: [illegible] CHIEF, TOP. ENG'G BRANCH

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EMBANKMENT AND EXCAVATION
STABILITY ANALYSIS
STEADY SEEPAGE CASE
STA. 95+00

APPROVED: Charles L. Hipp, CHIEF, ENGINEERING DIVISION
DATE: DEC 1968
SCALE: AS SHOWN
SPEC. NO.
APPROVED: [illegible] COL. C. E. DISTRICT ENGINEER
DRAWING NUMBER
SHEET

CHATFIELD RE-ALLOCATION STUDY PLATE B-4



PARTIAL POOL AND STEADY SEEPAGE – "S" STRENGTHS

OUTLET WORKS STA. 104 + 35

SCALE: 1 INCH = 50 FEET

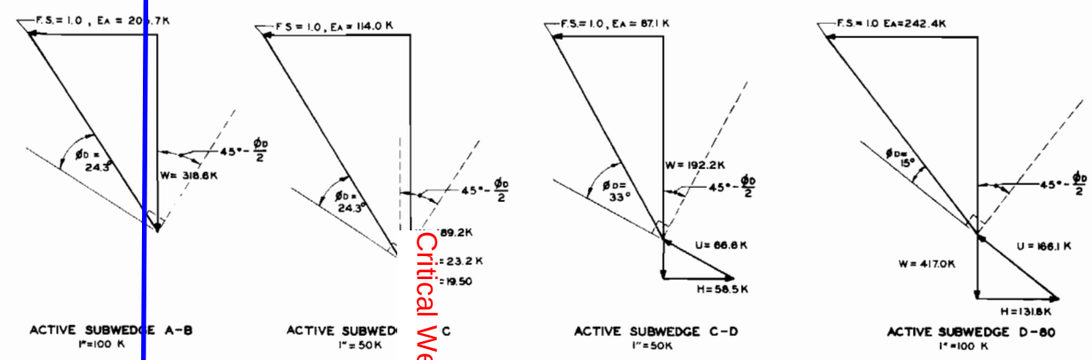
50' 0 50'

FACTORS OF SAFETY UPSTREAM PARTIAL POOL – "S" STRENGTH										
EL. OF FAILURE PLANE	ACTIVE WEDGE	PASSIVE WEDGE	FACTORS OF SAFETY POOL ELEVATION							
			5465		5450		5435		5410	
			NORM	E.Q.	NORM	E.Q.	NORM	E.Q.	NORM	E.Q.
5353	80	400	1.495	0.88	1.460	0.90	*1.456	*0.93	1.67	1.08
5320	100	400	1.57	0.90	1.51	0.91	—	—	1.64	1.05
Req'd. F.S.			1.50	1.0						

* Critical

FACTORS OF SAFETY DOWNSTREAM STEADY SEEPAGE – "S" STRENGTH				
EL. OF FAILURE PLANE	ACTIVE WEDGE	PASSIVE WEDGE	FACTORS OF SAFETY	
			NORMAL	E.Q.
5353	60	600	1.62	0.96
5320	60	600	1.58 *	0.92 *
Req'd. F.S.			1.50	1.0

* Critical



ACTIVE SUBWEDGE A–B
1" = 100 K

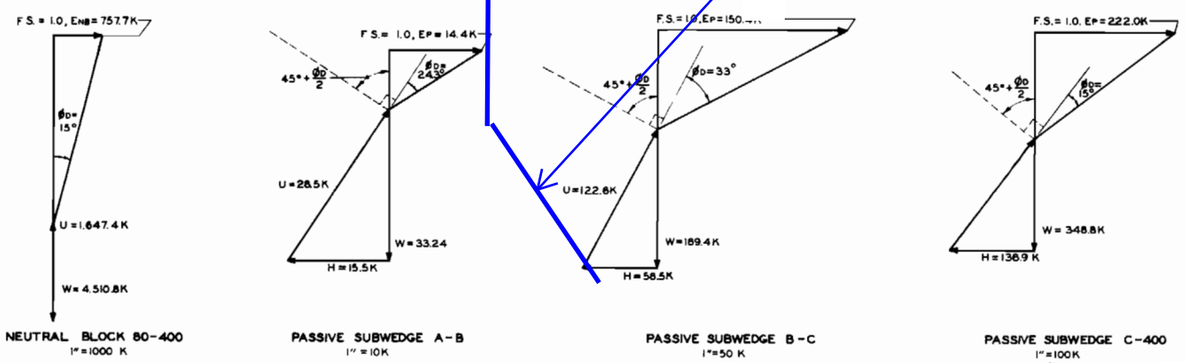
ACTIVE SUBWEDGE B–C
1" = 50 K

ACTIVE SUBWEDGE C–D
1" = 50K

ACTIVE SUBWEDGE D–80
1" = 100 K

ACTIVE WEDGE FORCE DIAGRAMS

Critical Wedge



NEUTRAL BLOCK 80–400
1" = 1000 K

PASSIVE SUBWEDGE A–B
1" = 10K

PASSIVE SUBWEDGE B–C
1" = 50 K

PASSIVE SUBWEDGE C–400
1" = 100K

NEUTRAL BLOCK AND PASSIVE WEDGE FORCE DIAGRAMS

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

ADOPTED SHEAR STRENGTH										
MATERIAL	IN PLACE WEIGHT		SAT'D WEIGHT		SUBM. WEIGHT		CONSOLIDATED DRAINED – "S"			
	#/FT³	T/FT³	#/FT³	#/FT³	#/FT³	T/FT³	TAN Ø	Ø	COH. T/SF	
EMBANKMENT										
Impervious	120.0	0.060	126.0	0.063	63.6	.0318	0.45	24.3	0.0	
Random	120.0	0.060	126.0	0.063	63.6	.0318	0.45	24.3	0.0	
Pervious	130.0	0.065	136.0	0.068	73.6	.0368	0.73	36.1	0.0	
Dawson FM	120.0	0.060	126.0	0.065	63.6	.0318	0.41	22.3	0.0	
FOUNDATION										
Clay			118.0	0.059	55.6	.0278	0.38	20.8	0.0	
Sand			125.0	0.063	62.6	.0313	0.65	33.0	0.0	
Dawson	130.0		135.0	0.0675	72.6	.0363	0.27*	15.0*	0.0*	

* Used in lieu of "S" strength

NOMENCLATURE

W = WEIGHT OF WEDGE OR BLOCK
H = NET HYDROSTATIC FORCE
U = UPLIFT ON FAILURE PLANE
C = (COHESION + TRIAL F.S.) × LENGTH OF PLANE
$Ø_D$ = ARCTAN TAN Ø ÷ TRIAL F.S.
F.S. = SHEAR STRENGTH ASSUMED / SHEAR STRENGTH NEEDED FOR EQUILIBRIUM

Note:
1. For notes refer to Plate B-73

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

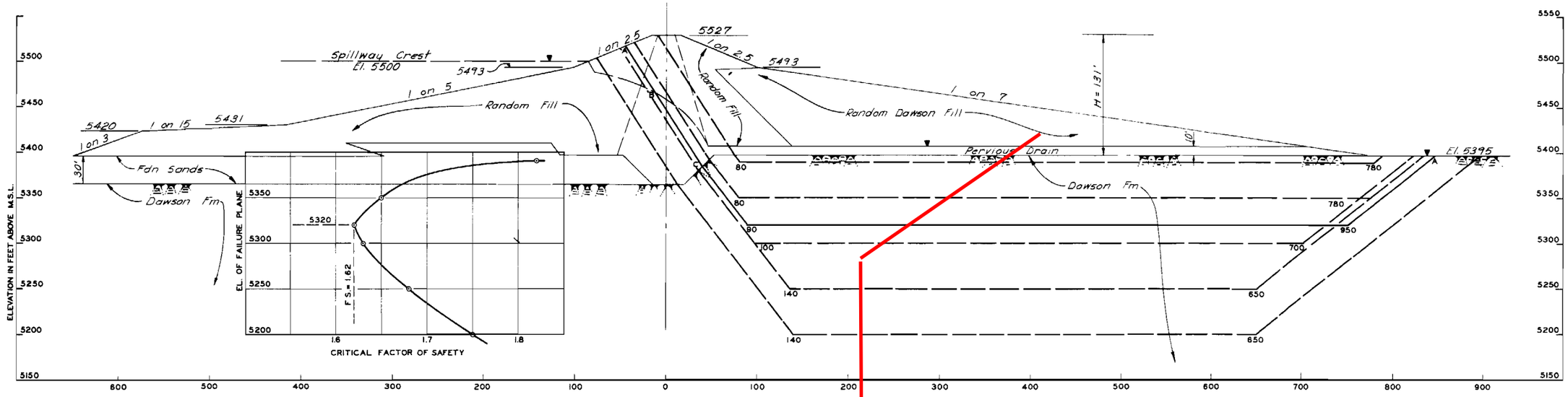
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: L G S
DRAWN BY: J F P
TRACED BY:
CHECKED BY: L G S
SUBMITTED BY: [illegible] CHIEF, EMBANKMENT & FDN. SEC.
APPROVED: [illegible] CHIEF, STN. & MTLS. BRANCH

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EMBANKMENT AND EXCAVATION
STABILITY ANALYSIS
PARTIAL POOL AND STEADY SEEPAGE CASES
OUTLET WORKS - STA. 104+35

APPROVED: Charles L. Hipp, CHIEF ENGINEERING DIVISION
DATE: DEC 1968
APPROVED: [illegible] COL., C. E., DISTRICT ENGINEER
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET

CHATFIELD RE-ALLOCATION STUDY PLATE B-5



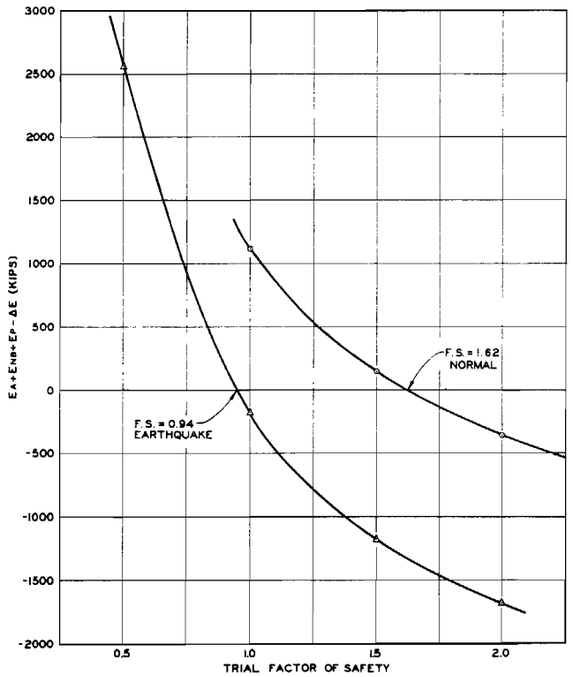
STEADY SEEPAGE-"S" STRENGTHS

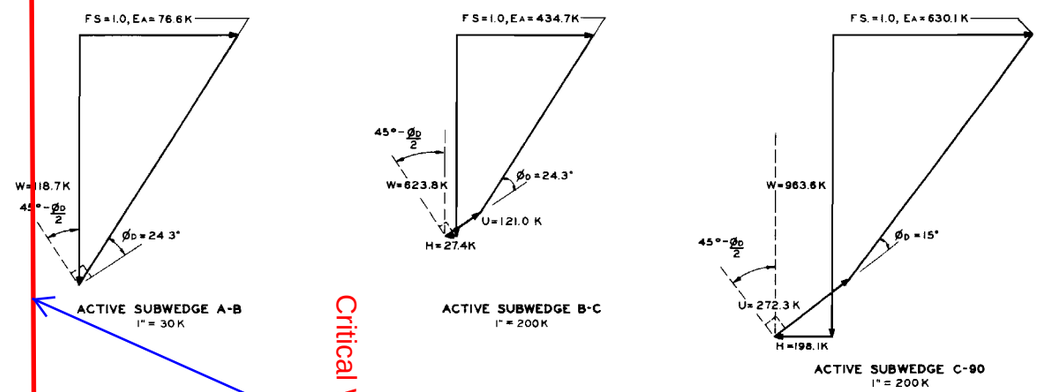
RT. VALLEY-STA. 68+50

SCALE: 1 INCH = 50 FEET

EL. OF FAILURE PLANE	ACTIVE WEDGE	PASSIVE WEDGE	FACTOR OF SAFETY NORM.	FACTOR OF SAFETY E.Q
5390	80	780	1.82	1.12
5350	80	780	1.65	0.97
5320	90	750	1.62 *	0.94 *
5300	100	700	1.63	0.94
5250	140	650	1.68	0.97
5200	140	650	1.75	0.98
Reqd. Factor of Safety			1.50	1.0

*Critical

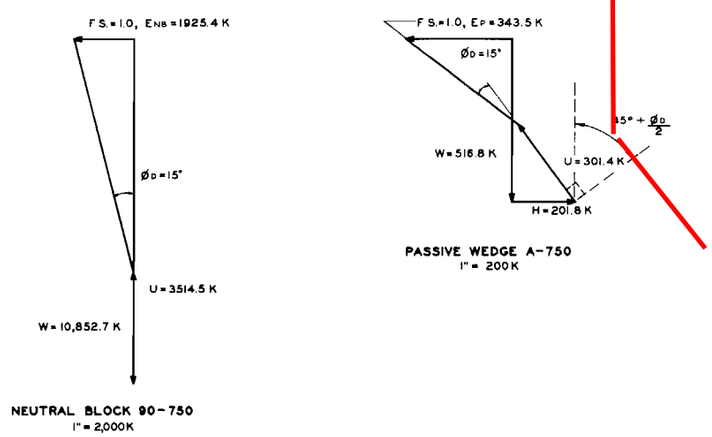




Critical Wedge

ACTIVE WEDGE FORCE DIAGRAMS

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.



NEUTRAL BLOCK AND PASSIVE WEDGE FORCE DIAGRAMS

NOMENCLATURE

W = WEIGHT OF WEDGE OR BLOCK
H = NET HYDROSTATIC FORCE
U = UPLIFT ON FAILURE PLANE
C = (COHESION + TRIAL F.S.) × LENGTH OF PLANE
ϕ_D = ARCTAN TAN ϕ ÷ TRIAL F.S.
F.S. = SHEAR STRENGTH ASSUMED / SHEAR STRENGTH NEEDED FOR EQUILIBRIUM

NOTE:
1 For NOTES, see Plate B-73

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

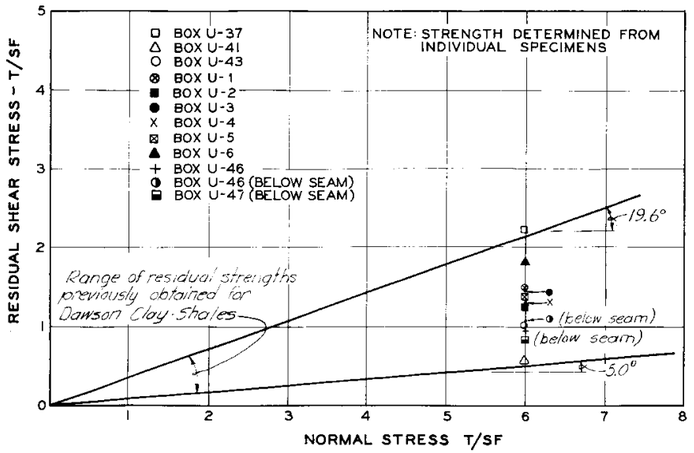
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: L G S
DRAWN BY: E J G
TRACED BY:
CHECKED BY: L G S
SUBMITTED BY: CHIEF, DESIGN FOUNDATION & MATERIALS SECTION

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EMBANKMENT AND EXCAVATION
STABILITY ANALYSIS
STEADY SEEPAGE CASE
RIGHT VALLEY-STA. 68+50

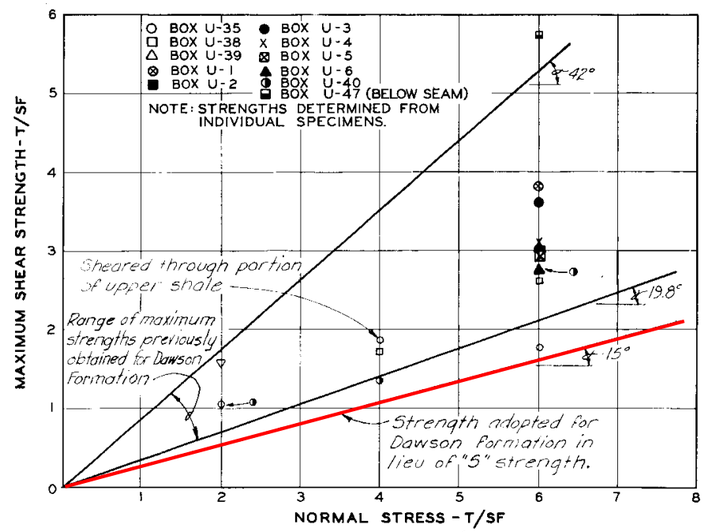
APPROVED: CHIEF FDN & MTLS BRANCH
APPROVED: Charles L. Hipp, CHIEF ENGINEERING DIVISION
DATE: DEC. 1966
APPROVED: COL. C. E. DISTRICT ENGINEER
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.



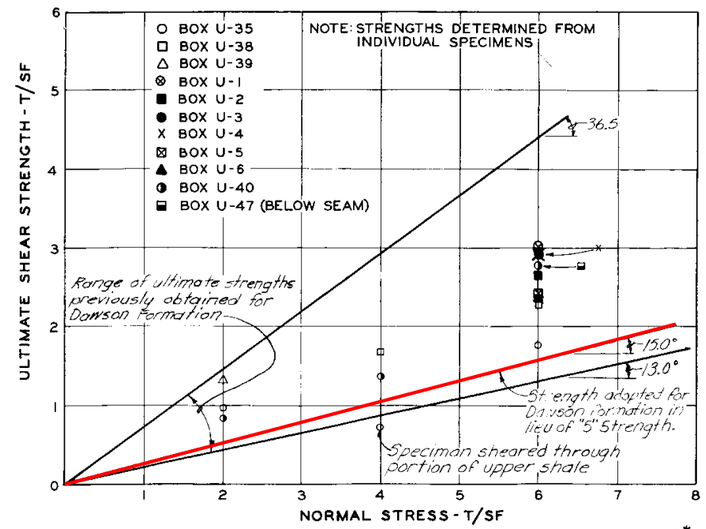
SUMMARY OF RESIDUAL DIRECT SHEAR TESTS ON SOFT SEAM

FIGURE 1



SUMMARY OF DIRECT SHEAR TESTS (MAXIMUM STRENGTH) ON SOFT SEAM MATERIAL.

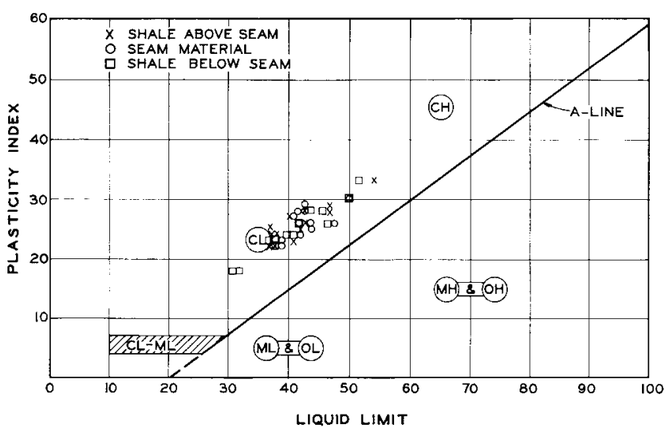
FIGURE 2



SUMMARY OF DIRECT SHEAR TESTS (ULTIMATE STRENGTH)* ON SOFT SEAM MATERIAL.

FIGURE 3

*NOTE: ULTIMATE STRENGTH IS THAT AFTER HORIZONTAL DEFORMATION OF 0.5 INCH IN DIRECT SHEAR.



PLASTICITY CHART

FIGURE 4

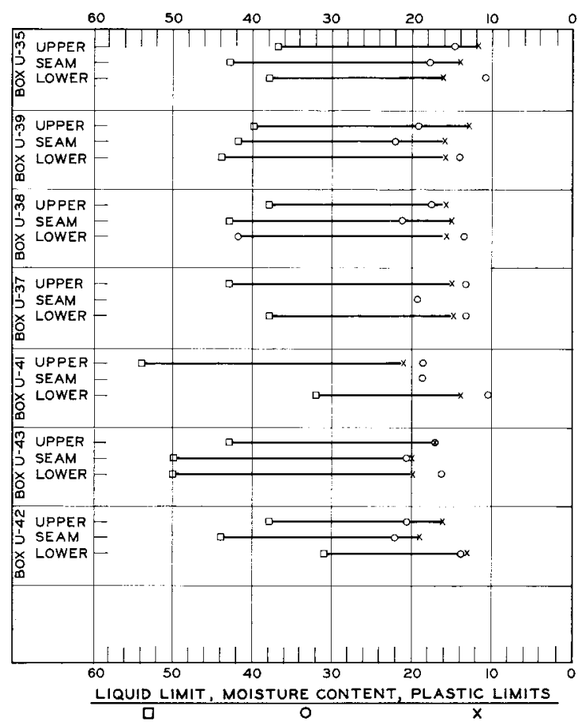


FIGURE 5

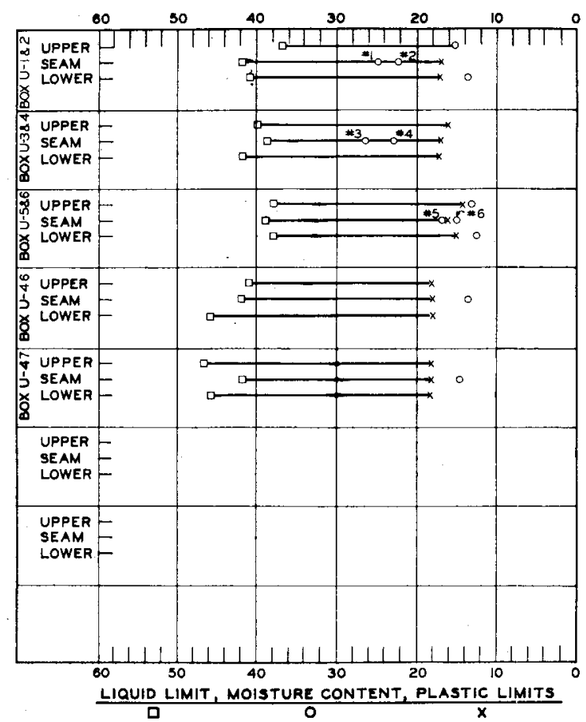


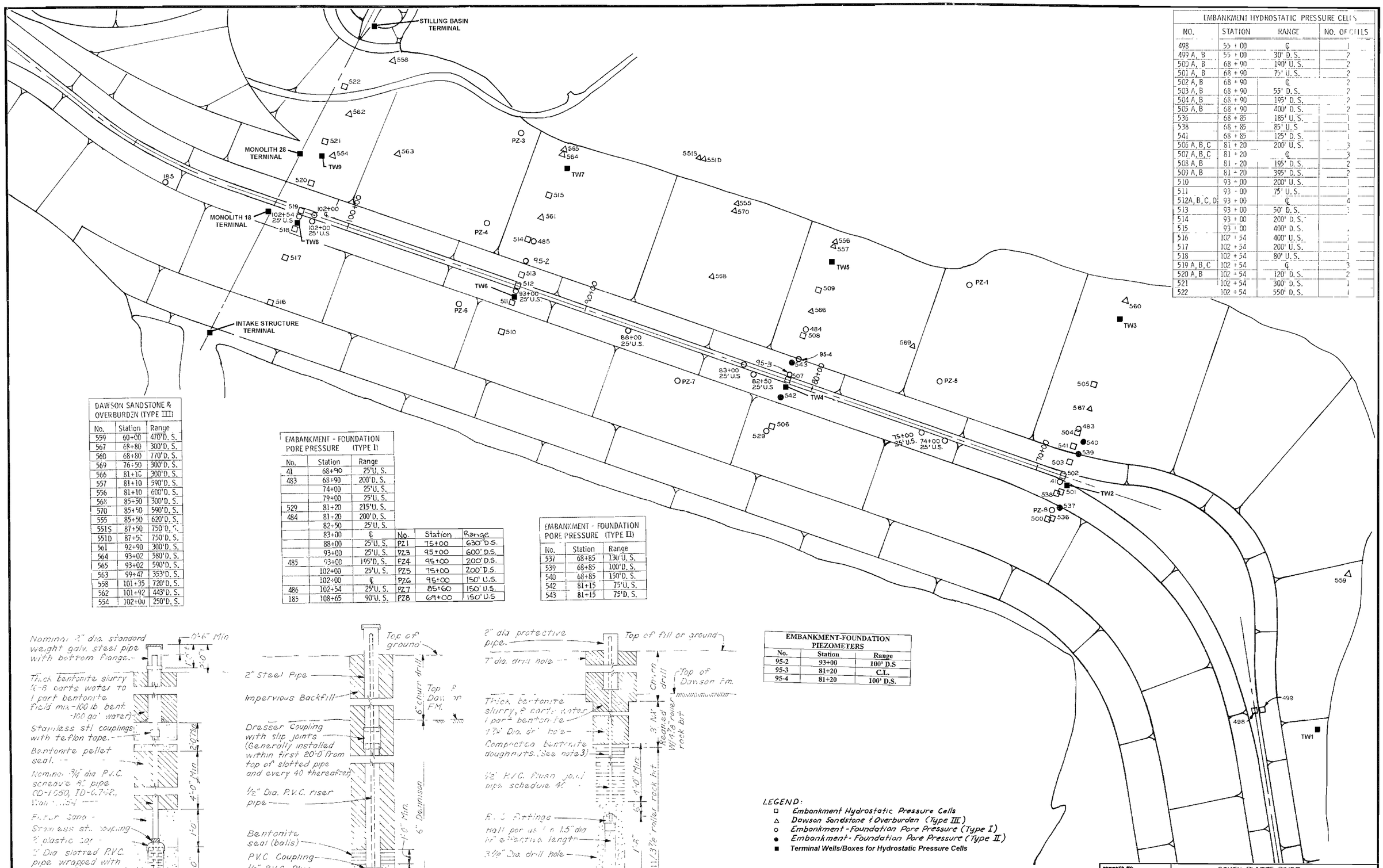
FIGURE 6

MOISTURE AND ATTERBERG LIMITS RELATIONSHIPS

NOTE:

1. Residual Tests on Boxes U-46 and U-47 were performed on precut samples; all other tests were performed on undisturbed samples.

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
EMBANKMENT AND EXCAVATION
UNDISTURBED DAWSON FORMATION
SUMMARIES OF DIRECT SHEAR TESTS
(RESIDUAL AND NORMAL) ON SOFT SEAM
U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA.
OCT. 1970



EMBANKMENT HYDROSTATIC PRESSURE CELLS			
NO.	STATION	RANGE	NO. OF CELLS
498	55 + 00	℄	1
499 A, B	55 + 00	30' D. S.	2
500 A, B	68 + 90	190' U. S.	2
501 A, B	68 + 90	75' U. S.	2
502 A, B	68 + 90	℄	2
503 A, B	68 + 90	55' D. S.	2
504 A, B	68 + 90	195' D. S.	2
505 A, B	68 + 90	400' D. S.	2
536	68 + 85	185' U. S.	1
538	68 + 85	85' U. S	1
541	68 + 85	125' D. S.	1
506 A, B, C	81 + 20	200' U. S.	3
507 A, B, C	81 + 20	℄	3
508 A, B	81 + 20	195' D. S.	2
509 A, B	81 + 20	395' D. S.	2
510	93 + 00	200' U. S.	1
511	93 + 00	75' U. S.	1
512A, B, C, D	93 + 00	℄	4
513	93 + 00	50' D. S.	1
514	93 + 00	200' D. S.	1
515	93 + 00	400' D. S.	1
516	102 + 54	400' U. S.	1
517	102 + 54	200' U. S.	1
518	102 + 54	80' U. S.	1
519 A, B, C	102 + 54	℄	3
520 A, B	102 + 54	120' D. S.	2
521	102 + 54	300' D. S.	1
522	102 + 54	550' D. S.	1

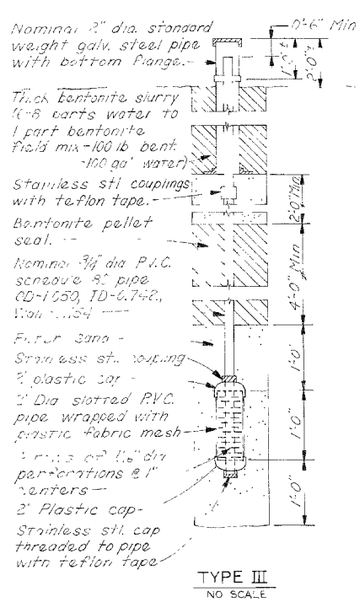
DAWSON SANDSTONE & OVERBURDEN (TYPE III)		
No.	Station	Range
559	60+00	470'D. S.
567	68+80	300'D. S.
560	68+80	770'D. S.
569	76+50	300'D. S.
566	81+10	300'D. S.
557	81+10	590'D. S.
556	81+10	600'D. S.
568	85+50	300'D. S.
570	85+50	590'D. S.
555	85+50	620'D. S.
551S	87+50	750'D. S.
551D	87+50	750'D. S.
561	92+90	300'D. S.
564	93+02	580'D. S.
565	93+02	590'D. S.
563	99+47	353'D. S.
558	101+35	720'D. S.
562	101+92	443'D. S.
554	102+00	250'D. S.

EMBANKMENT - FOUNDATION PORE PRESSURE (TYPE I)		
No.	Station	Range
41	68+90	25'U. S.
483	68+90	200'D. S.
	74+00	25'U. S.
	79+00	25'U. S.
529	81+20	215'U. S.
484	81+20	200'D. S.
	82+50	25'U. S.
	83+00	℄
	88+00	25'U. S.
	93+00	25'U. S.
485	93+00	195'D. S.
	102+00	25'U. S.
	102+00	℄
486	102+54	25'U. S.
185	108+65	90'U. S.

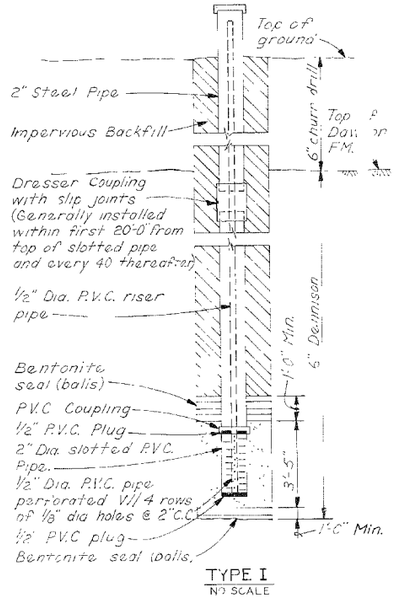
No.	Station	Range
PZ1	75+00	630'D.S.
PZ3	95+00	600' D.S.
PZ4	95+00	200'D.S.
PZ5	75+00	200'D.S.
PZ6	95+00	150' U.S.
PZ7	85+60	150' U.S.
PZ8	69+00	150' U.S

EMBANKMENT - FOUNDATION PORE PRESSURE (TYPE II)		
No.	Station	Range
537	68+85	130'U. S.
539	68+85	100'D. S.
540	68+85	150'D. S.
542	81+15	75'U. S.
543	81+15	75'D. S.

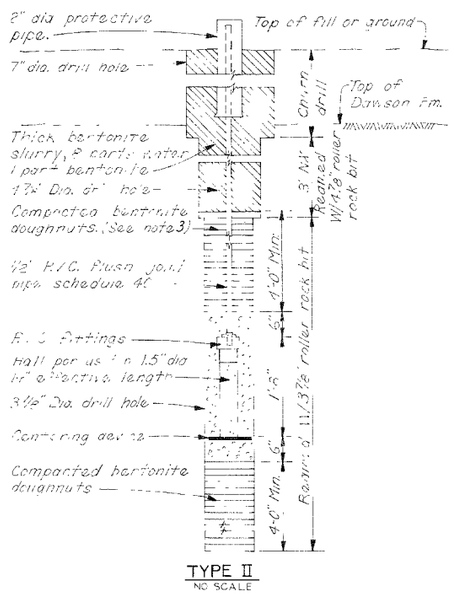
EMBANKMENT-FOUNDATION PIEZOMETERS		
No.	Station	Range
95-2	93+00	100' D.S
95-3	81+20	C.L.
95-4	81+20	100' D.S.



TYPE III
NO SCALE



TYPE I
NO SCALE

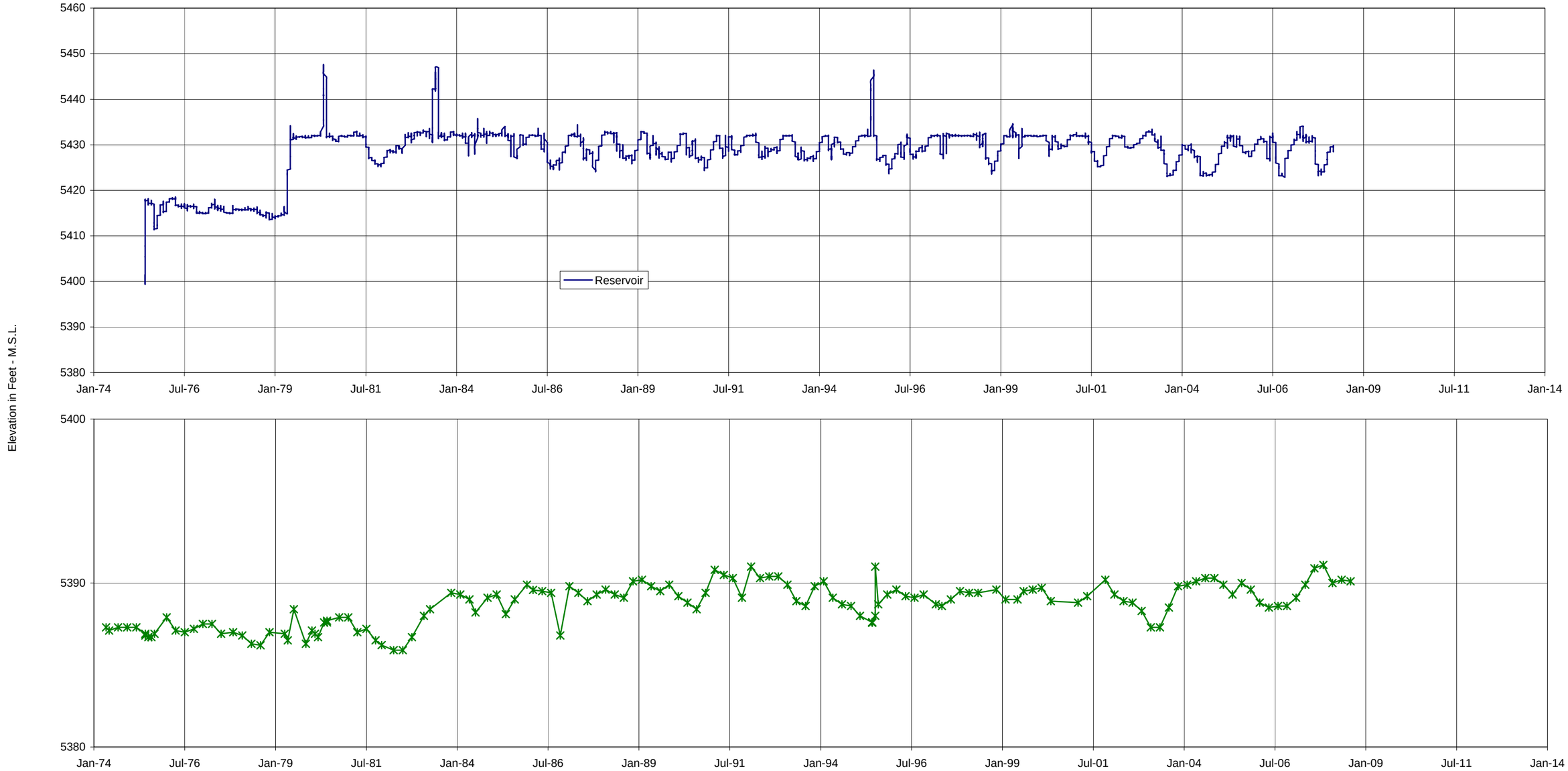


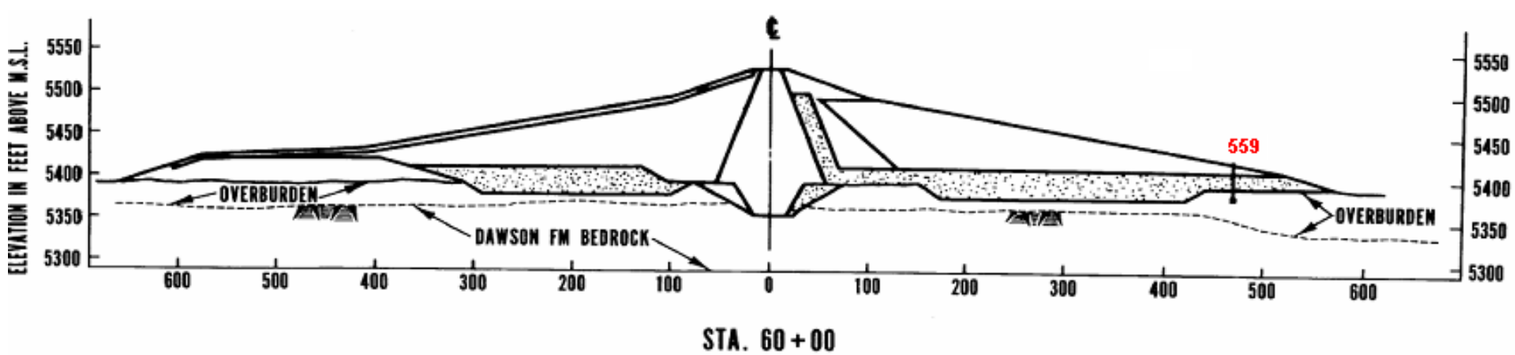
TYPE II
NO SCALE

LEGEND:

- □ Embankment Hydrostatic Pressure Cells
- △ Dawson Sandstone & Overburden (Type III)
- ○ Embankment-Foundation Pore Pressure (Type I)
- ● Embankment-Foundation Pore Pressure (Type II)
- ■ Terminal Wells/Boxes for Hydrostatic Pressure Cells

DESIGNED BY:	SOUTH PLATTE RIVER CHATFIELD LAKE, COLORADO EMBANKMENT PIEZOMETER LOCATION PLAN	
DRAWN BY:		
CHECKED BY:		
SUBMITTED BY: CHIEF SECTION		
RECOMMENDED: CHIEF DESIGN BRANCH	APPROVED: CHIEF ENGINEERING DIVISION	DATE:
APPROVED: COL. D. E., DISTRICT ENGINEER	SCALE: AS SHOWN	SPEC. NO. DACA45
	SHEET	DRAWING NUMBER

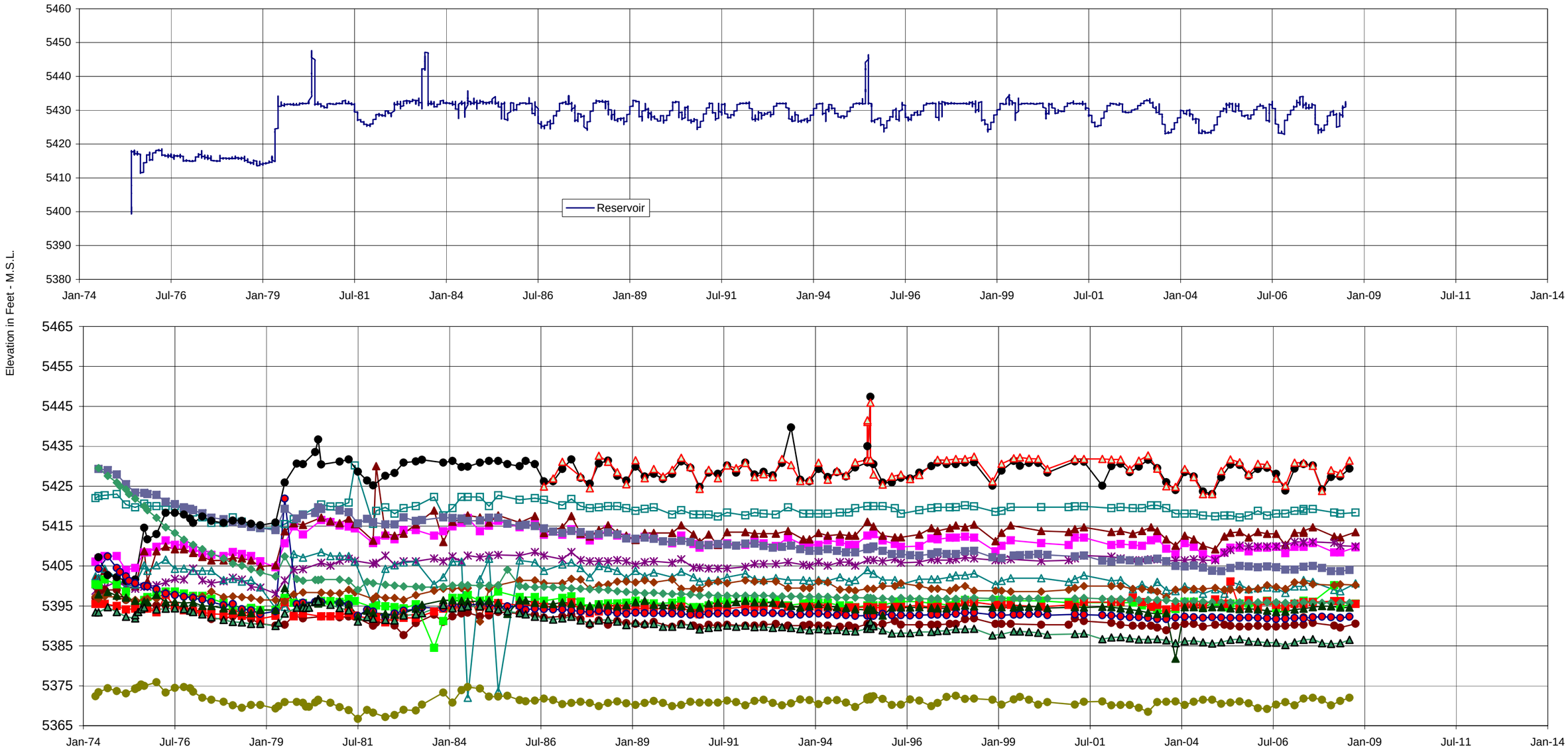


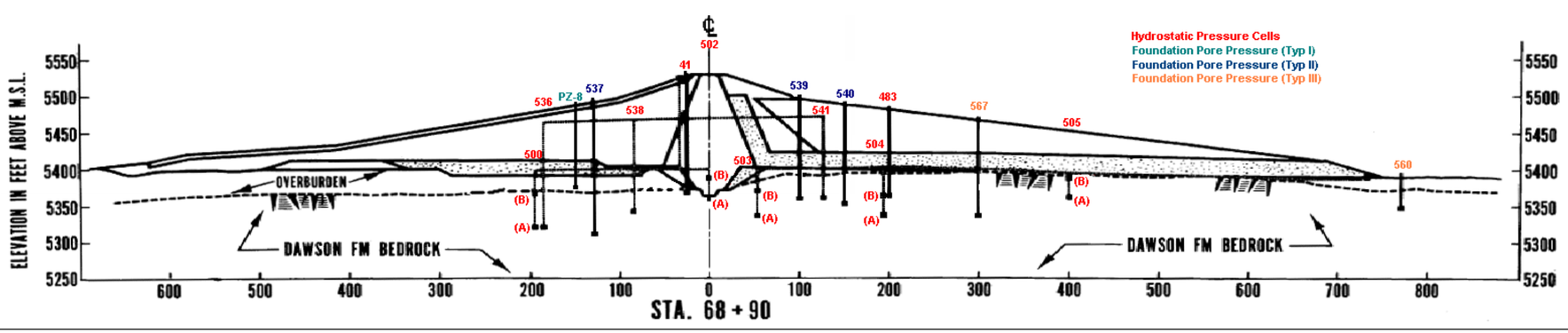


559

Chatfield Dam and Lake, Colorado
Main Embankment & Foundation Piezmeter Observations
Station 60+00

Re-Allocation Study Plate C-2

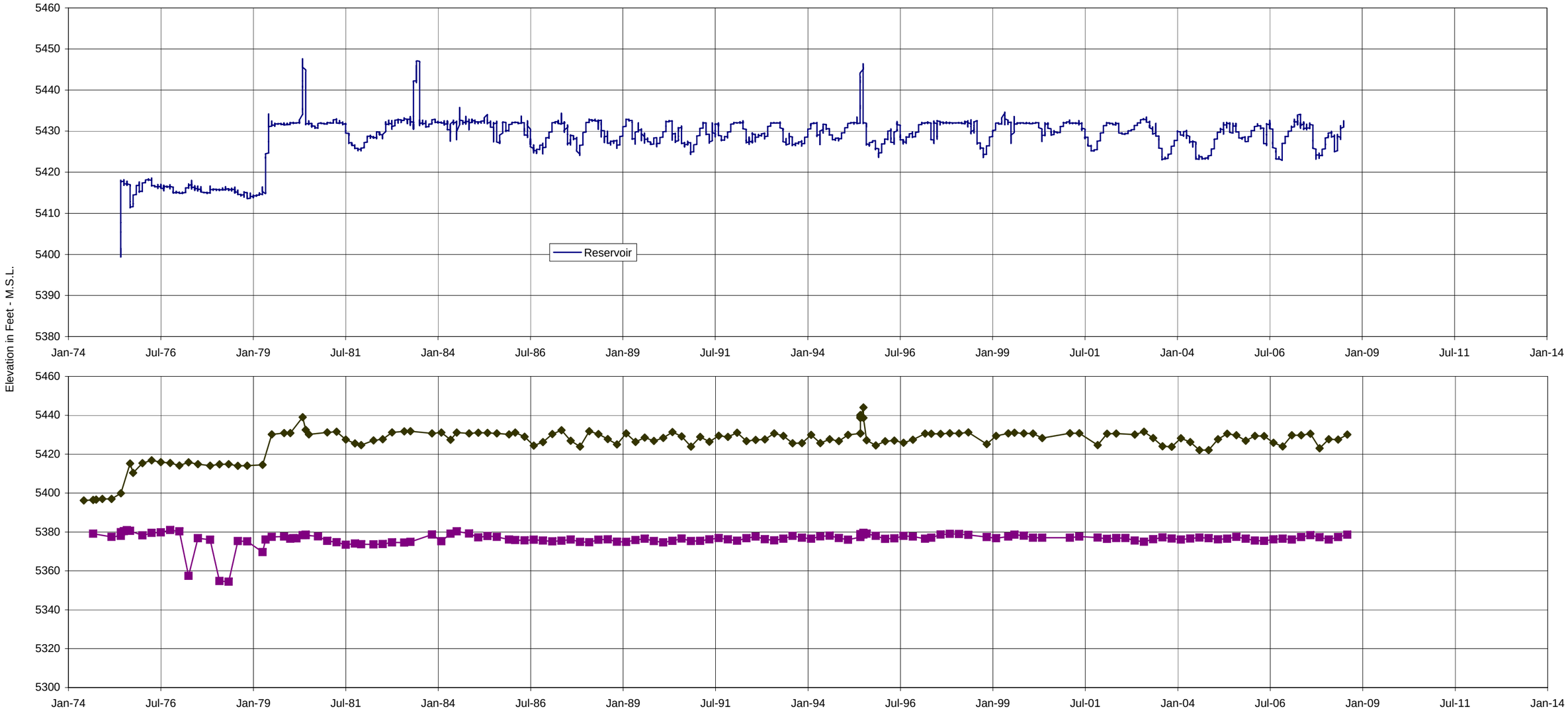


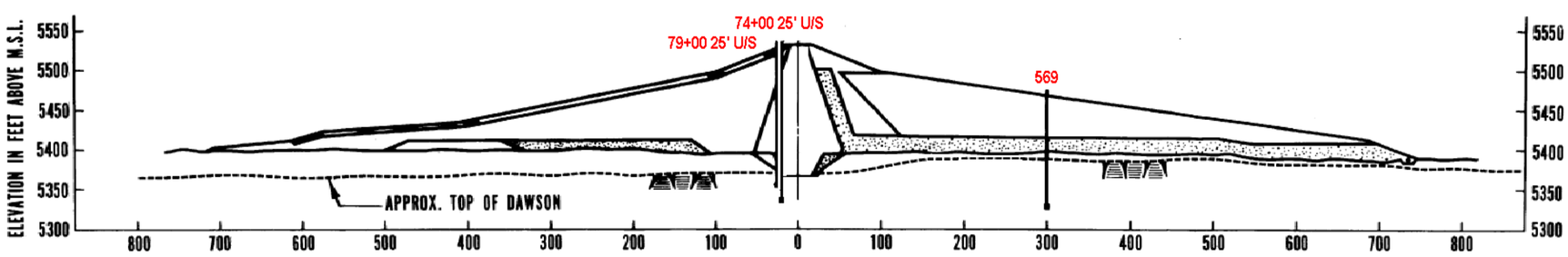


536 538A 502A 502B 541
504A 504B 505A 505B 537
41 539 540 483 567
560 PZ-8

Chatfield Dam and Lake, Colorado
Main Embankment & Foundation Piezmeter Observations
Station 68+90

Re-Allocation Study Plate C-3





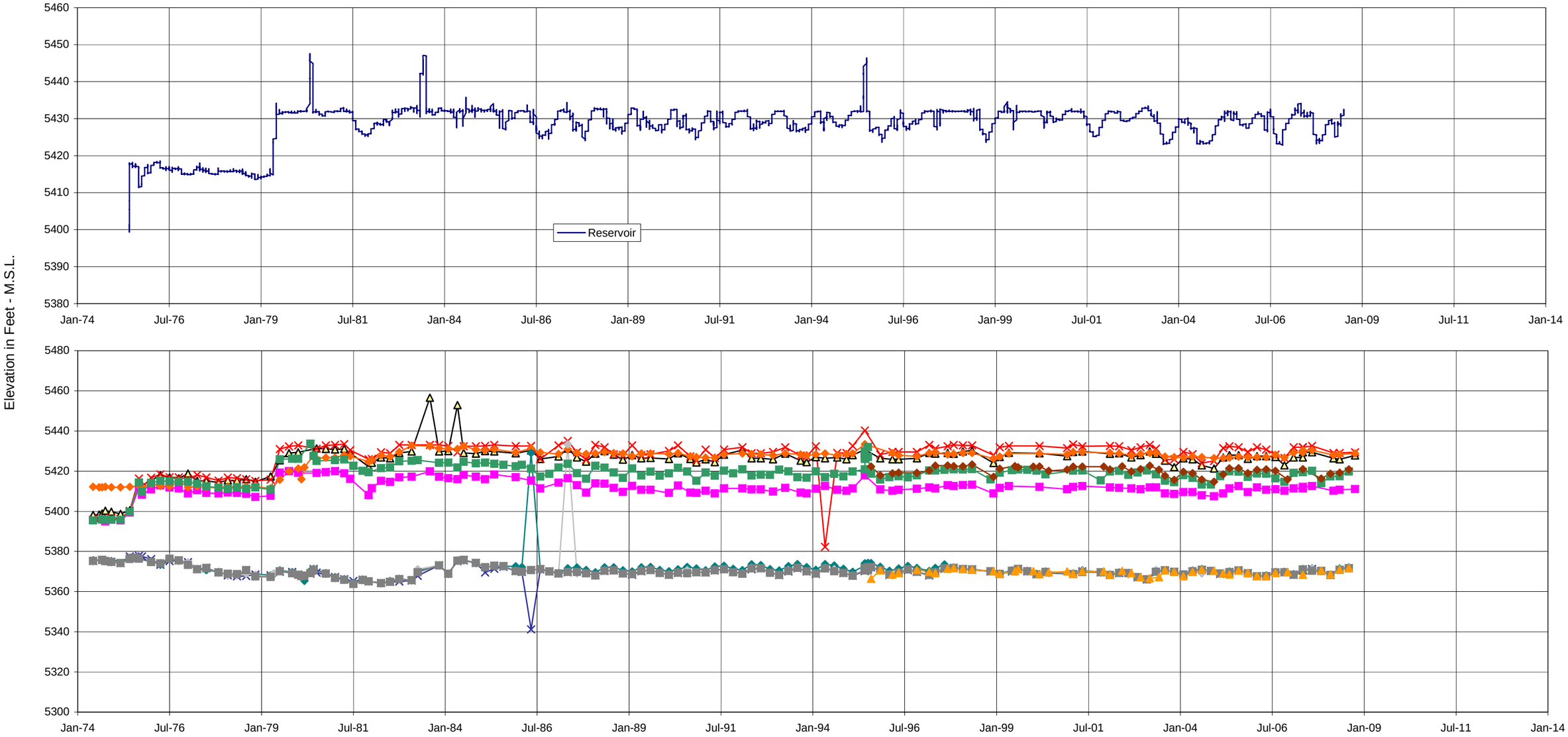
TYPICAL SECTION
STA. 74+00 TO STA. 79+00

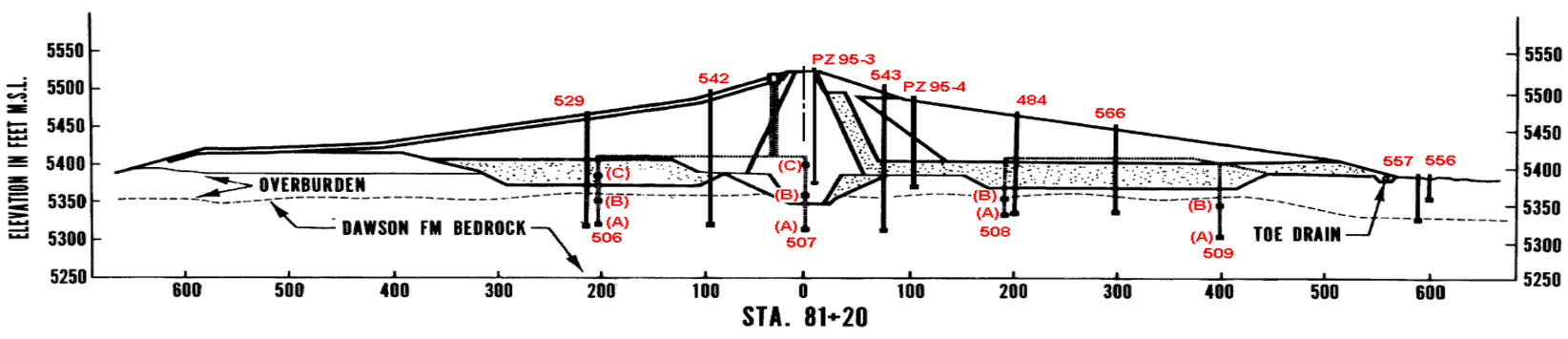
79+00/25US
569

NOTE: Piez 74+00 25' U/S was found plugged & was abandoned in July 1972

Chatfield Dam and Lake, Colorado
Main Embankment & Foundation Piezmeter Observations
Station 79+00

Re-Allocation Study Plate C-4

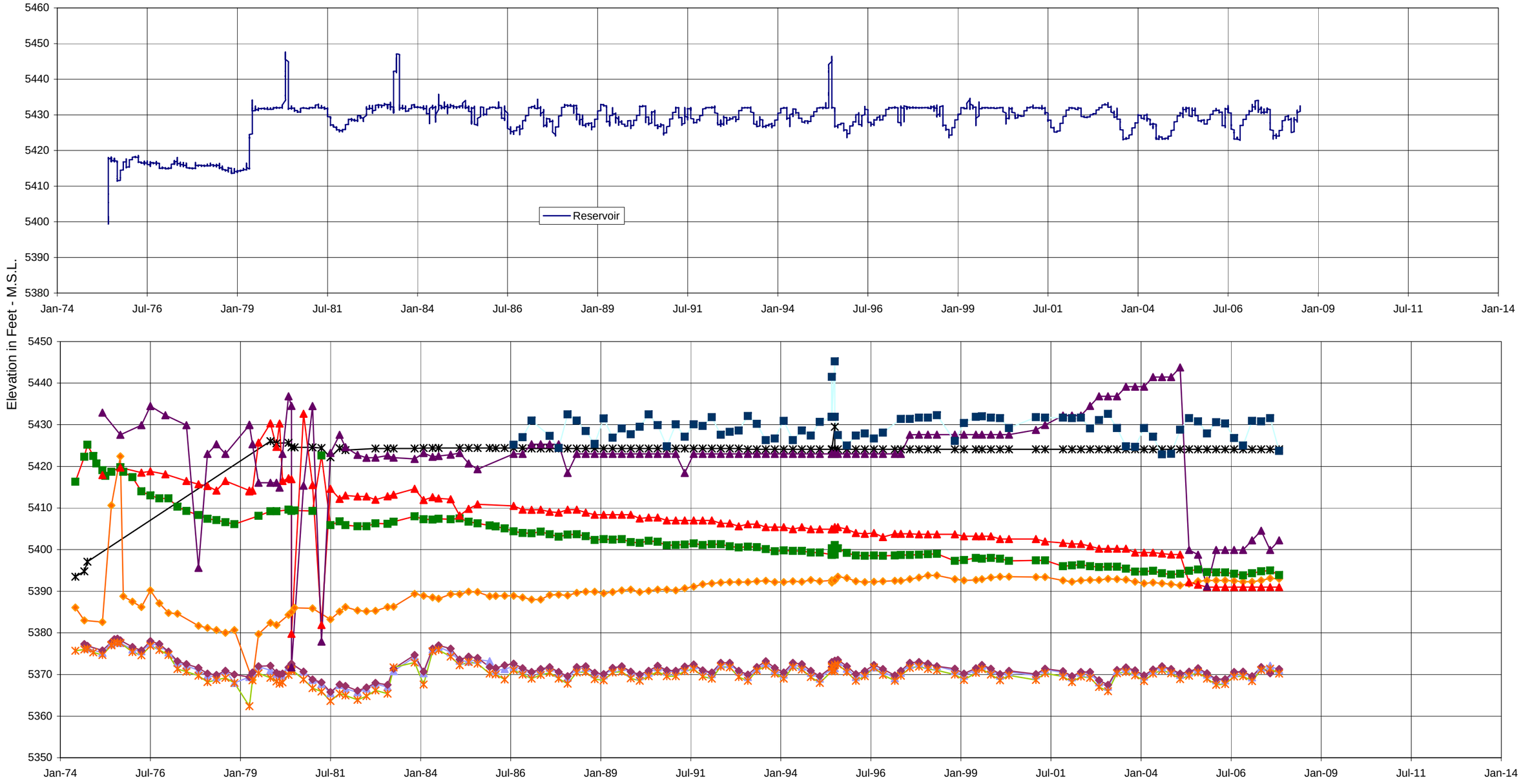


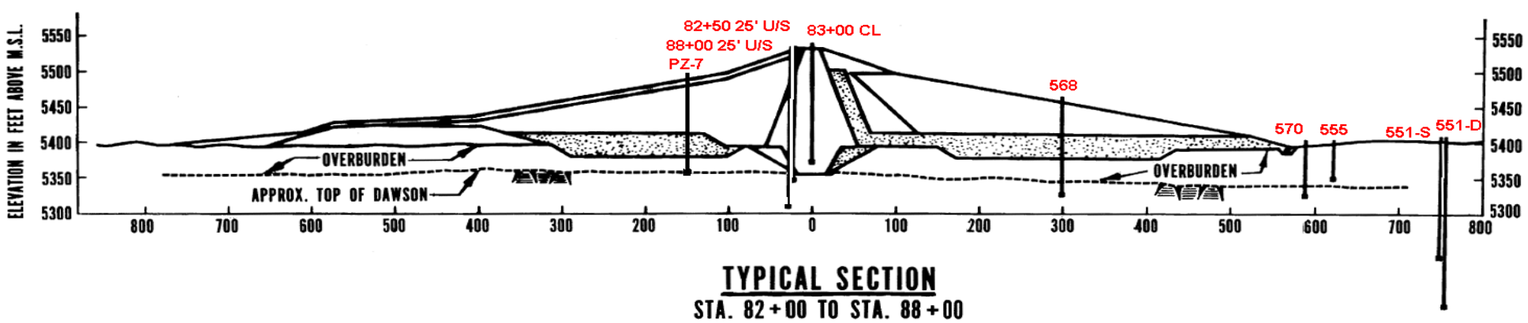


NOTE: Piez. 543 Reading discontinued Aug 1994 due to sounder cable broke and lodged in pipe.

Chatfield Dam and Lake, Colorado
Main Embankment & Foundation Piezmeter Observations
Station 81+20

Re-Allocation Study Plate C-5

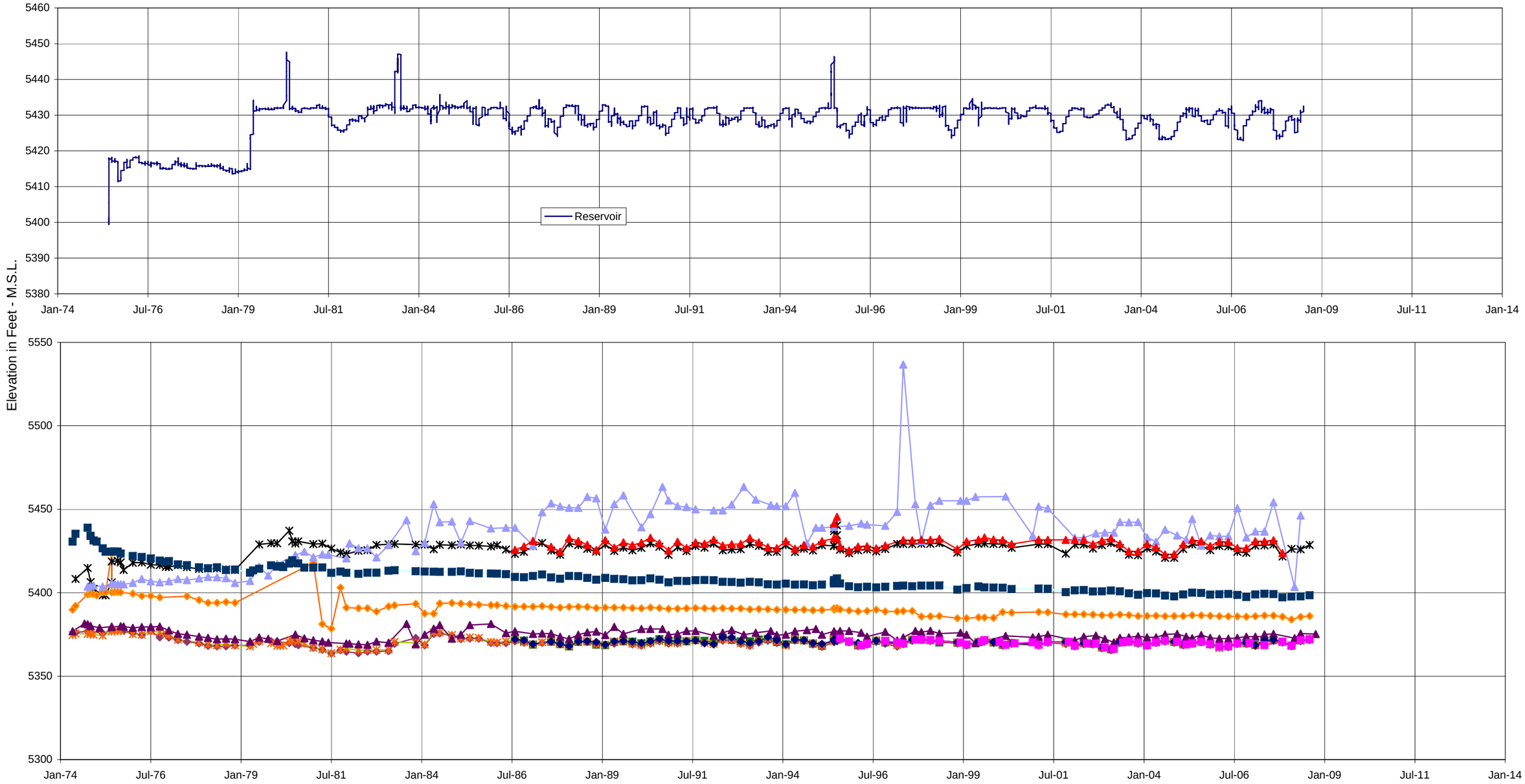


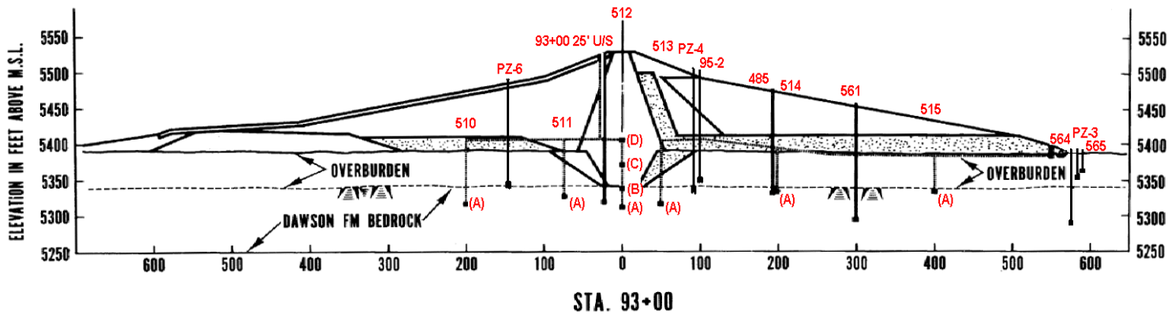


TYPICAL SECTION
STA. 82+00 TO STA. 88+00

Chatfield Dam and Lake, Colorado
Main Embankment & Foundation Piezmeter Observations
Station 82+00

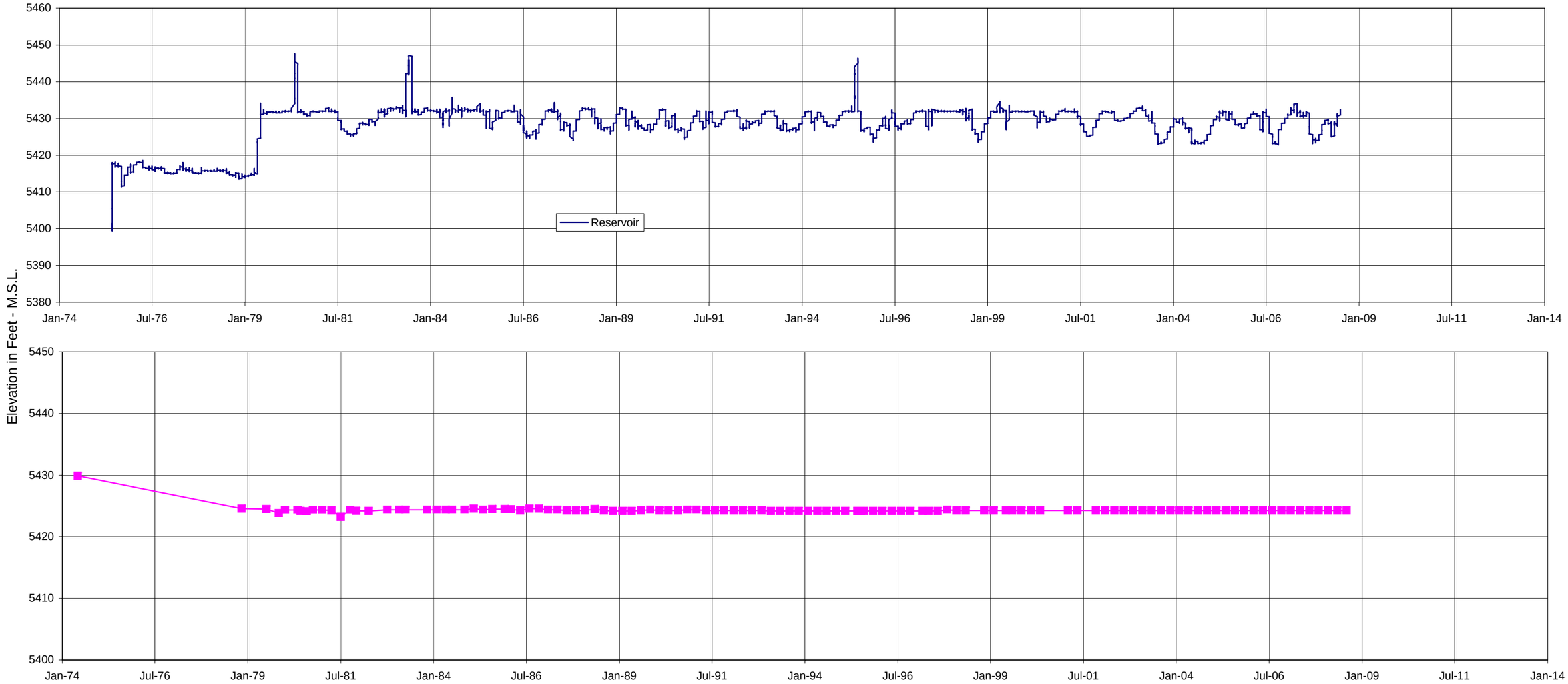
Re-Allocation Study Plate C-6

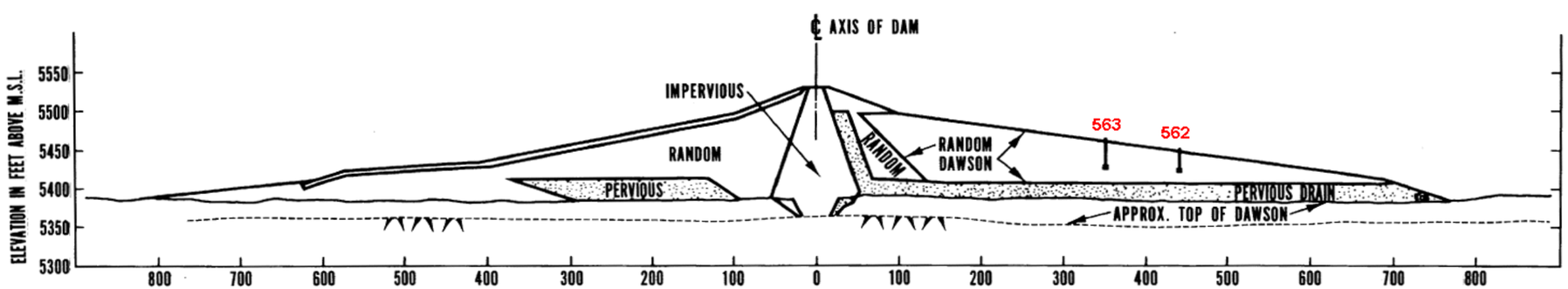




Chatfield Dam and Lake, Colorado
Main Embankment & Foundation Piezmeter Observations
Station 93+00

Re-Allocation Study Plate C-7

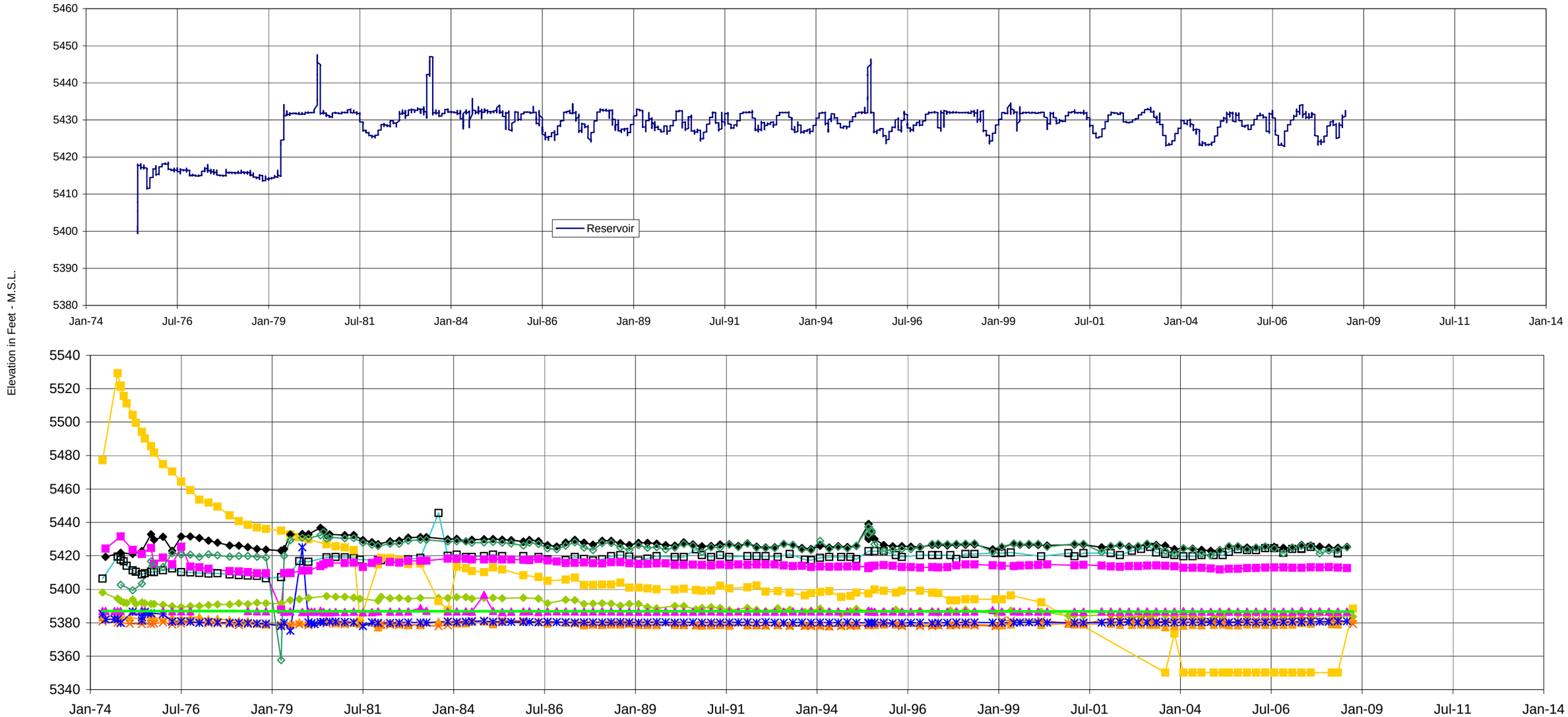


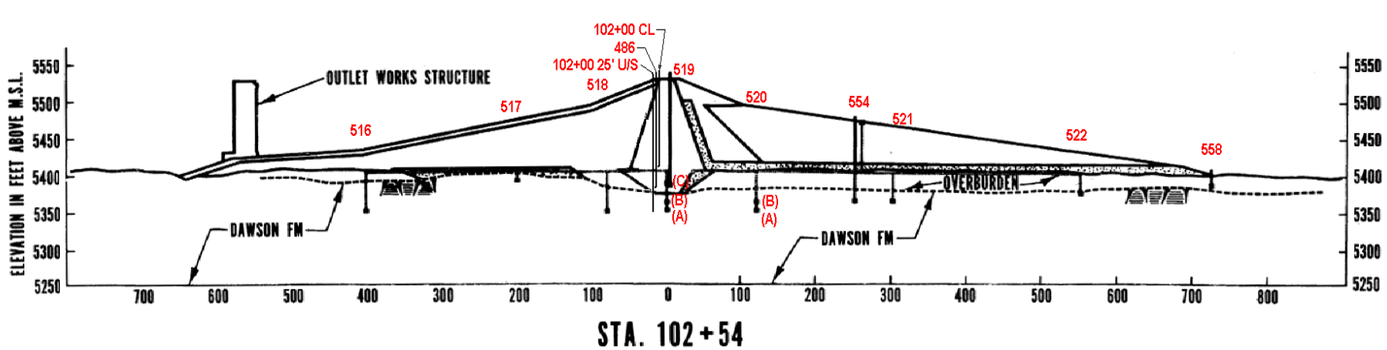


TYPICAL SECTION
STA. 99+00 TO STA. 102+00

Chatfield Dam and Lake, Colorado
Main Embankment & Foundation Piezmeter Observations
Station 99+00

Re-Allocation Study Plate C-8

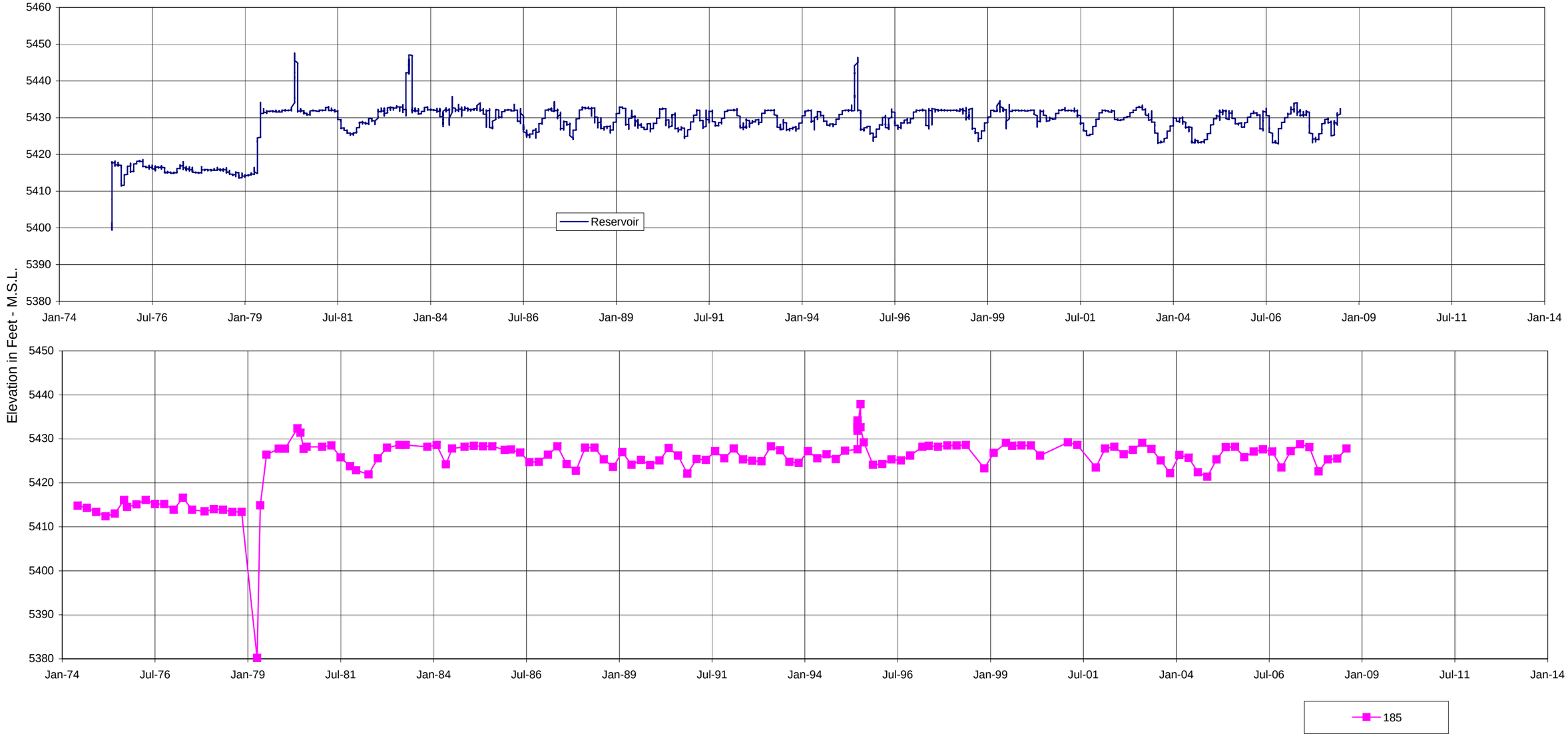


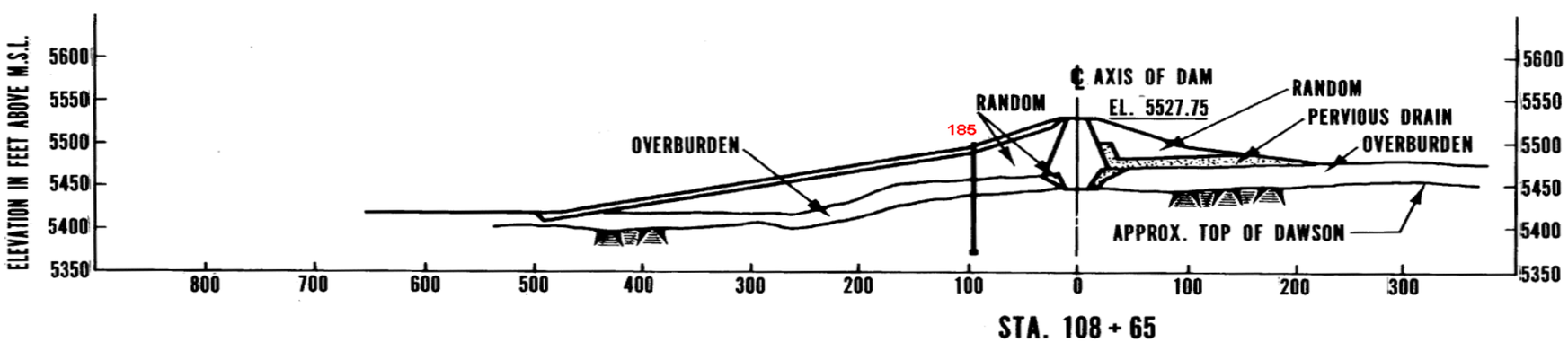


NOTE: Piez. 519A discontinued in May 1985 due to erratic behavior.
Piez. 519B discontinued in Aug 1995 due to difficulty in obtaining readings.
Piez 516 discontinued in Aug 1995 due to erratic behavior.

Chatfield Dam and Lake, Colorado
Main Embankment & Foundation Piezmeter Observations
Station 102+54

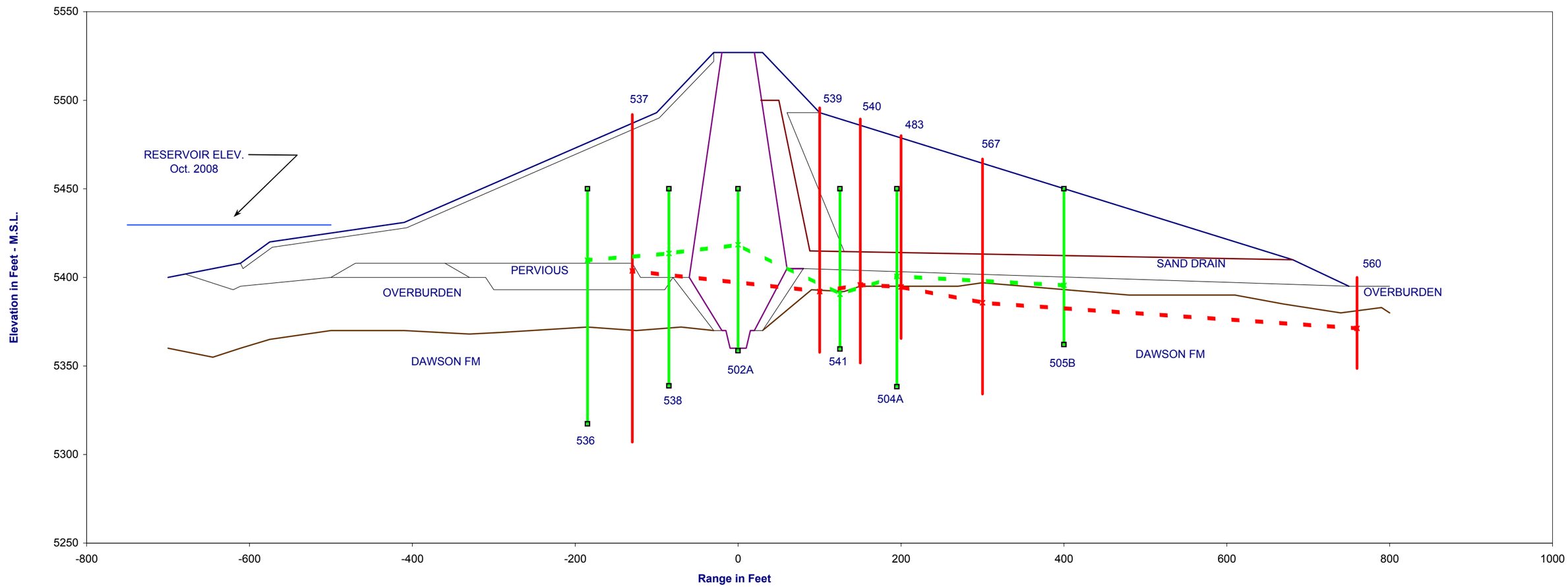
Re-Allocation Study Plate C-9





Chatfield Dam and Lake, Colorado
Main Embankment & Foundation Piezmeter Observations
Station 108+65

Re-Allocation Study Plate C-10



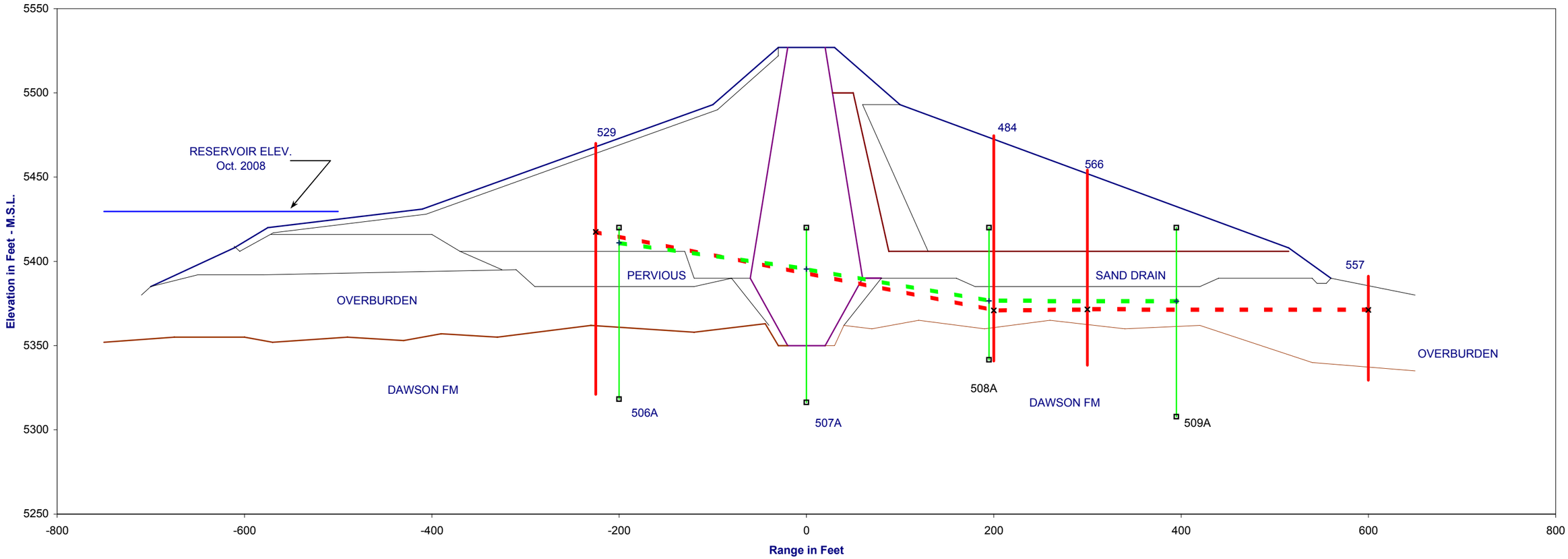
Embankment Station 68+80 to 68+90

Open Tube - May 2008 Data Res. El.(5431.15)

Pressure Cell - Oct 2008 Data Res. El.(5429.61)

Chatfield Dam and Lake, CO
Foundation Piezometer Observations
Station 68+80 to 68+90

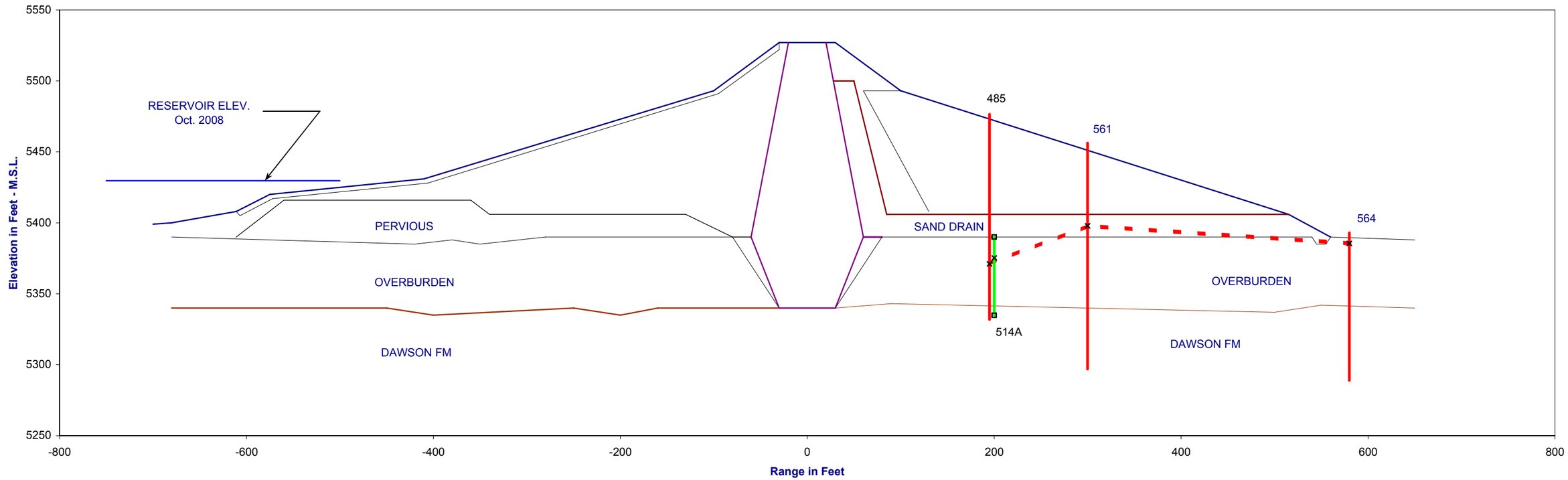
RE-ALLOCATION STUDY PLATE C-11



Embankment Station 81+10 to 81+20
Open Tube - May 2008 Data Res. El. (5431.15)
Pressure Cell - Oct 2008 Data Res. El.(5429.61)

Chatfield Dam and Lake, CO
Foundation Piezometer Observations
Station 81+10 to 81+20

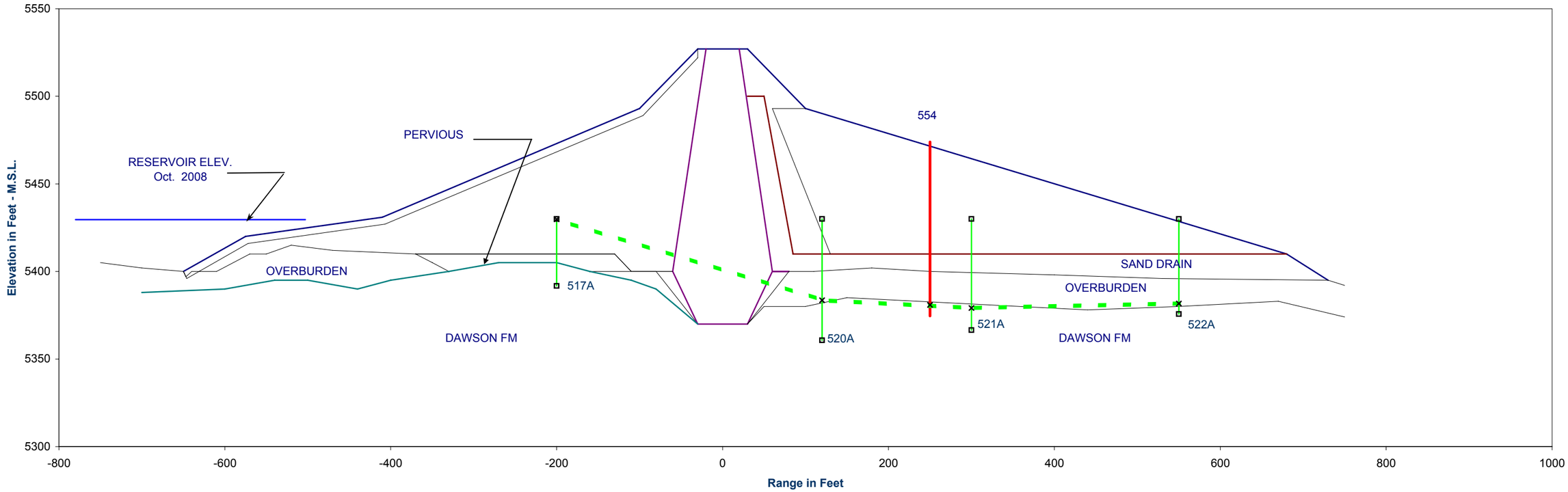
RE-ALLOCATION STUDY PLATE C-12



Embankment Station 92+90-93+02
Open Tube - May 2008 Data Res. El. (5431.15)
Pressure Cell - Oct 2008 Data Res. El.(5429.61)

Chatfield Dam and Lake, CO
Foundation Piezometer Observations
Station 93+00

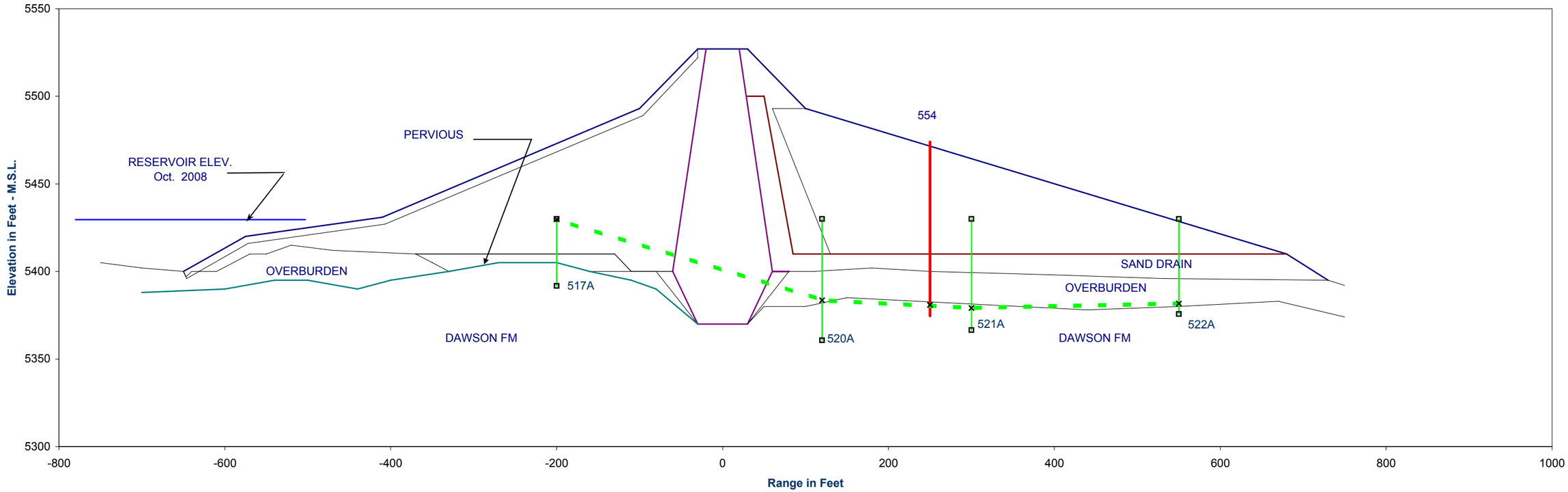
RE-ALLOCATION STUDY PLATE C-13



Embankment Station 101+35-102+00
Open Tube - May 2008 Data Res. El. (5431.15)
Pressure Cell - Oct 2008 Data Res. El.(5429.61)

Chatfield Dam and Lake, CO
Foundation Piezometer Observations
Station 101+35 to 102+00

RE-ALLOCATION STUDY PLATE C-14



Embankment Station 101+35-102+00
Open Tube - May 2008 Data Res. El. (5431.15)
Pressure Cell - Oct 2008 Data Res. El.(5429.61)

Chatfield Dam and Lake, CO
Foundation Piezometer Observations
Station 101+35 to 102+00

RE-ALLOCATION STUDY PLATE C-14

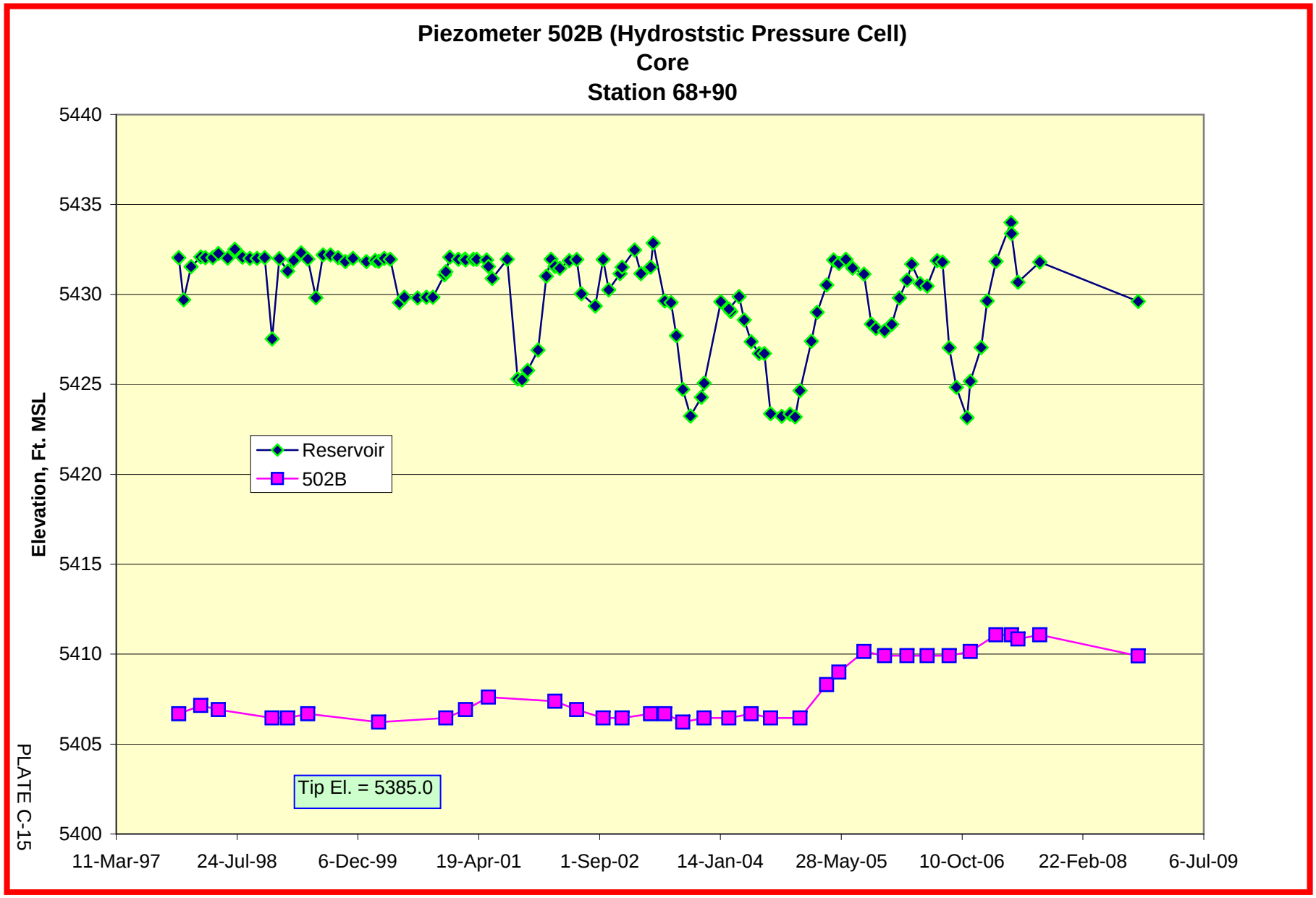

Piezometer 502B (Hydroststic Pressure Cell)
Core
Station 68+90
Elevation, Ft. MSL
5440
5435
5430
5425
5420
5415
5410
5405
5400
11-Mar-97
24-Jul-98
6-Dec-99
19-Apr-01
1-Sep-02
14-Jan-04
28-May-05
10-Oct-06
22-Feb-08
6-Jul-09
Reservoir
502B
Tip El. = 5385.0

PLATE C-15

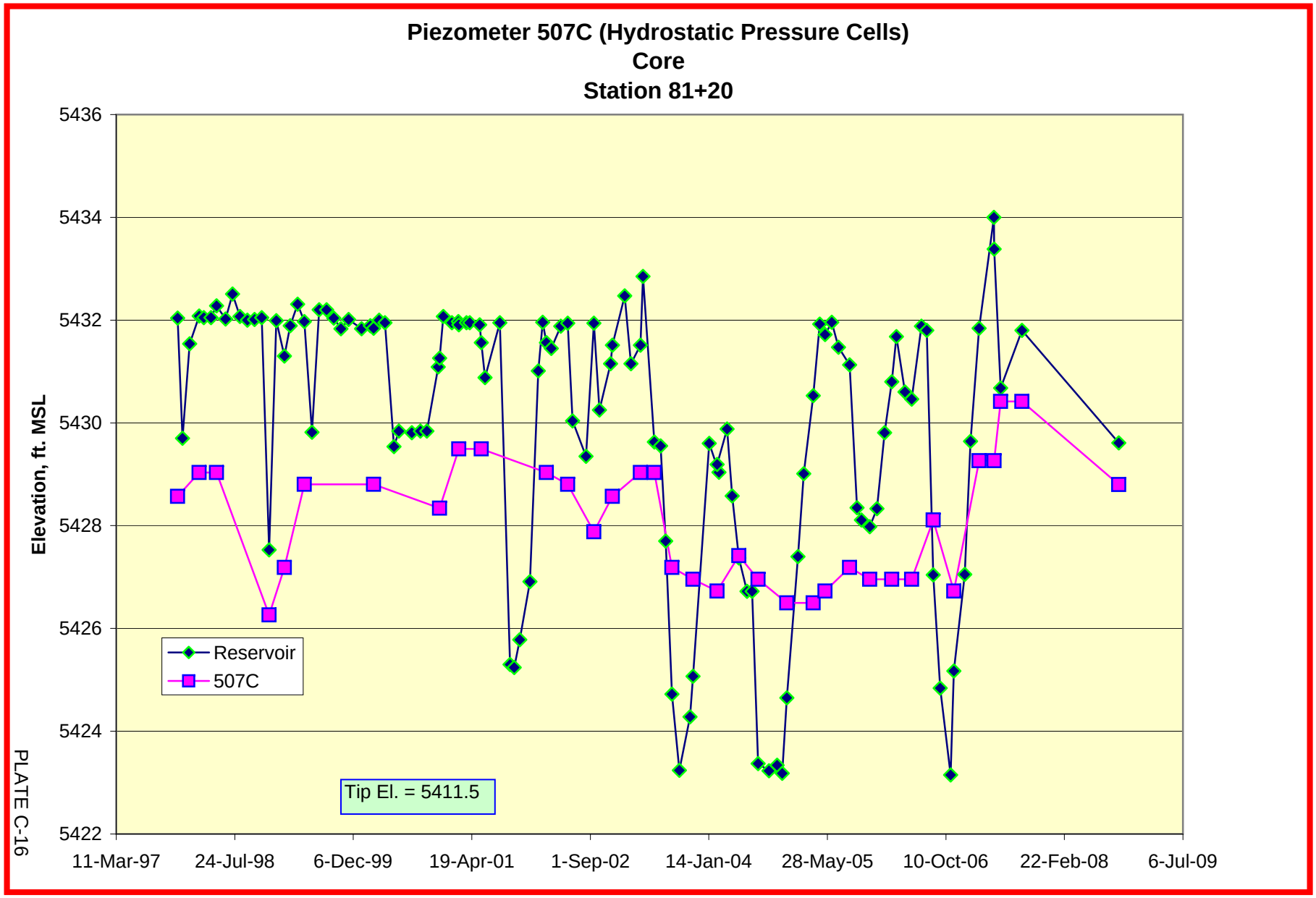
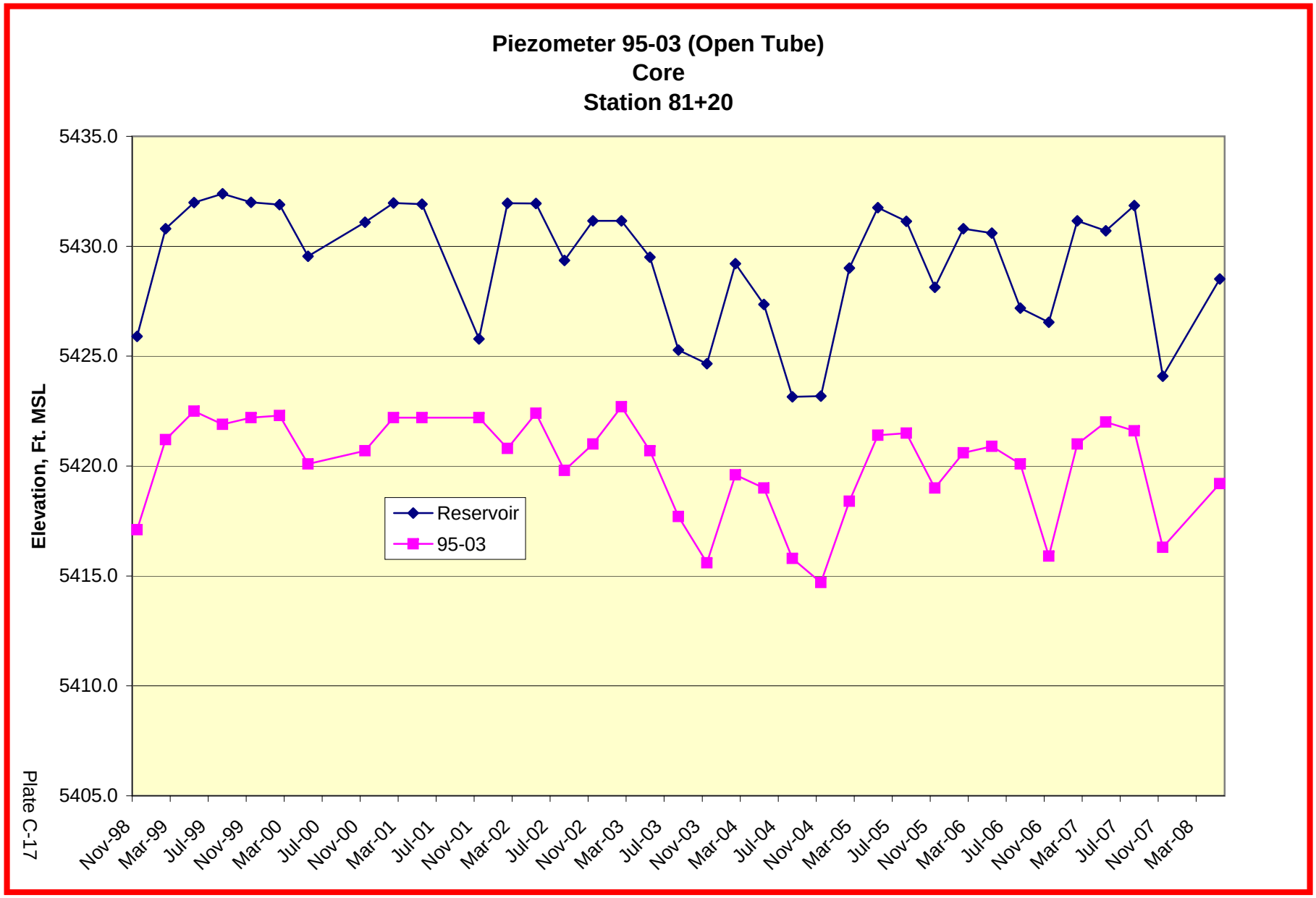

Piezometer 507C (Hydrostatic Pressure Cells)
Core
Station 81+20
Elevation, ft. MSL
5436
5434
5432
5430
5428
5426
5424
5422
11-Mar-97
24-Jul-98
6-Dec-99
19-Apr-01
1-Sep-02
14-Jan-04
28-May-05
10-Oct-06
22-Feb-08
6-Jul-09
Reservoir
507C
Tip El. = 5411.5

PLATE C-16


Piezometer 95-03 (Open Tube)
Core
Station 81+20
Elevation, Ft. MSL
5435.0
5430.0
5425.0
5420.0
5415.0
5410.0
5405.0
Reservoir
95-03
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
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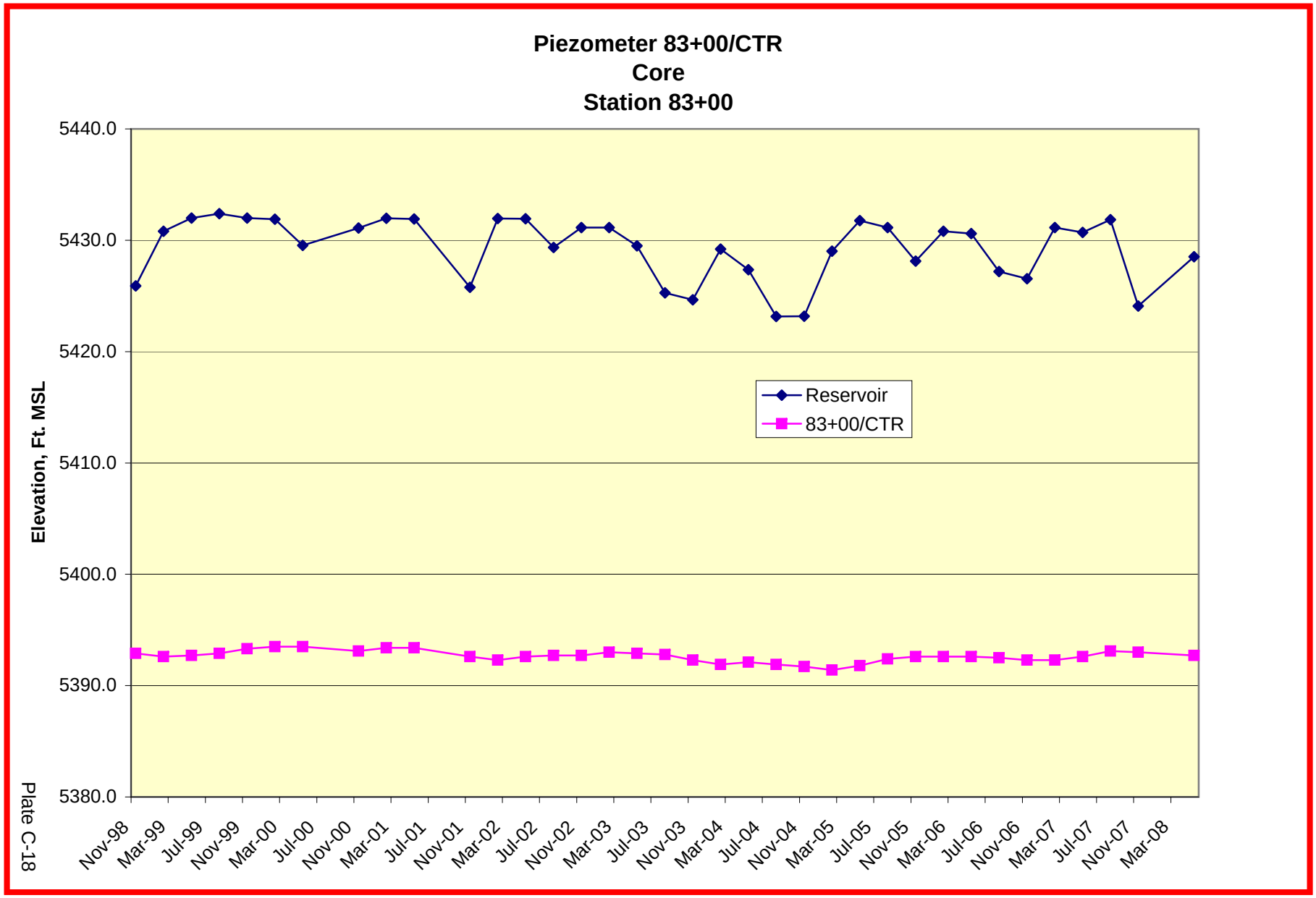
Mar-07
Jul-07
Nov-07
Mar-08


Piezometer 83+00/CTR
Core
Station 83+00
Elevation, Ft. MSL
5440.0
5430.0
5420.0
5410.0
5400.0
5390.0
5380.0
Reservoir
83+00/CTR
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
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Nov-05
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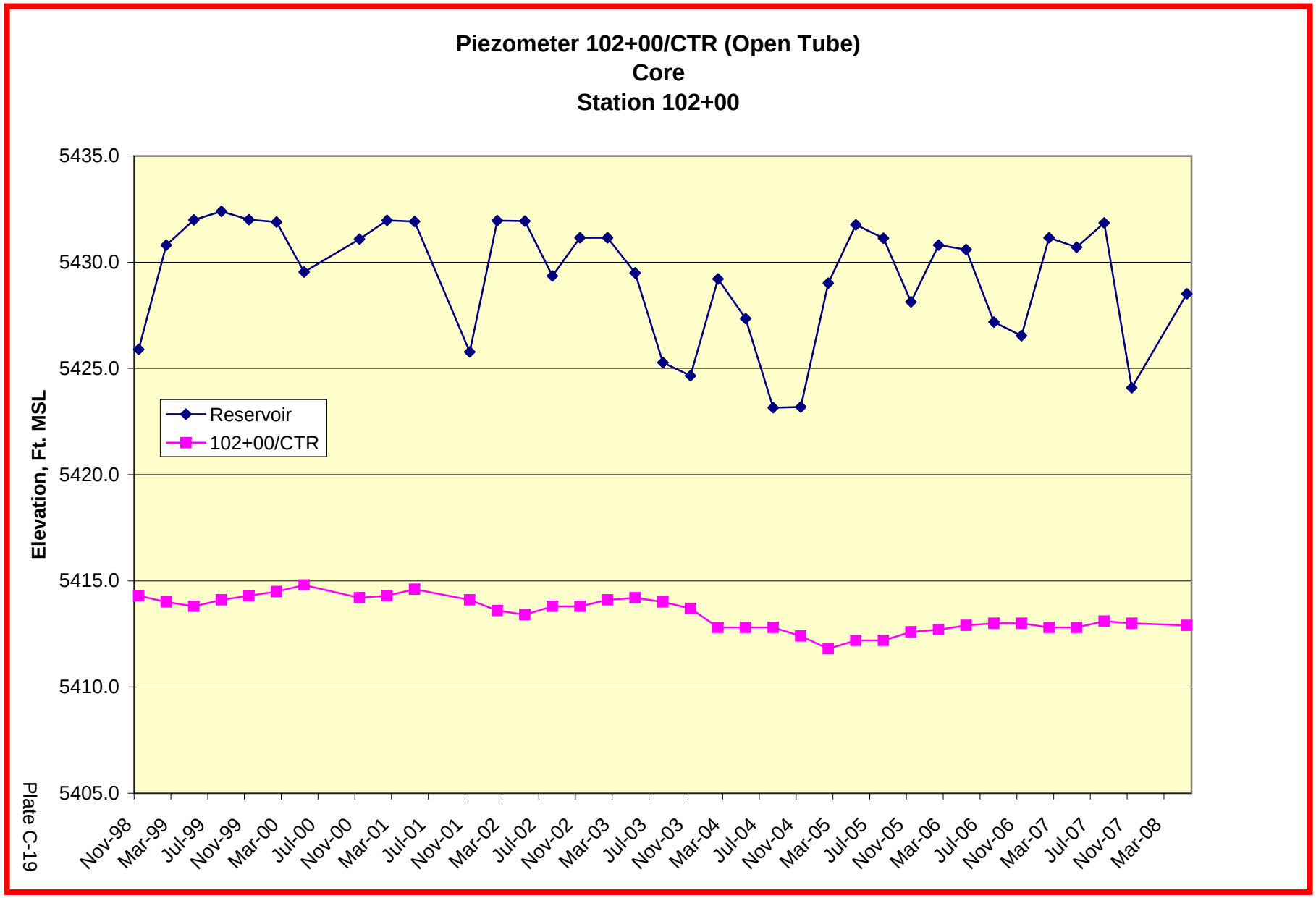
Jul-06
Nov-06
Mar-07
Jul-07
Nov-07
Mar-08


Piezometer 102+00/CTR (Open Tube)
Core
Station 102+00
Elevation, Ft. MSL
5435.0
5430.0
5425.0
5420.0
5415.0
5410.0
5405.0
Reservoir
102+00/CTR
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
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Nov-06
Mar-07
Jul-07
Nov-07
Mar-08

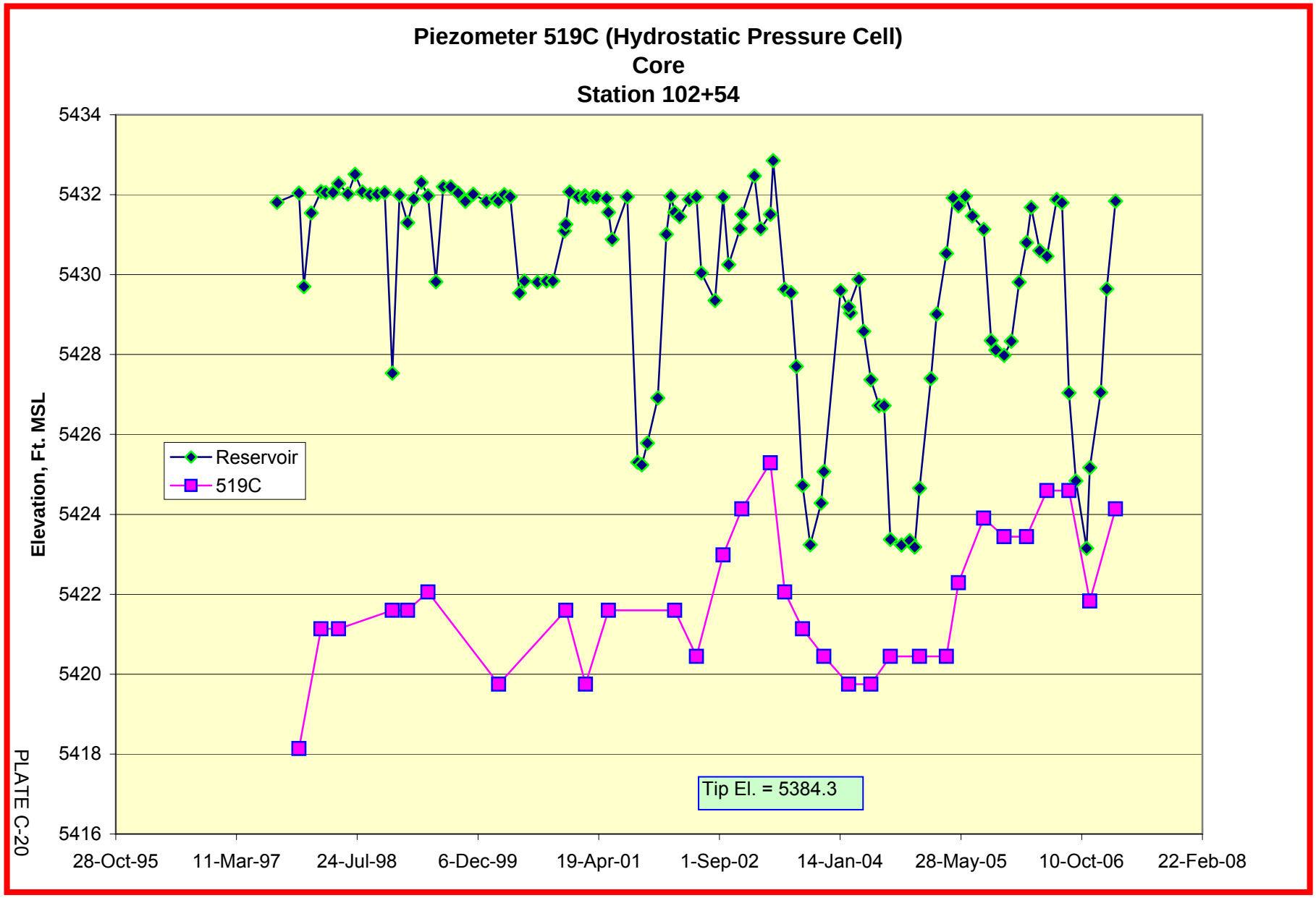

Piezometer 519C (Hydrostatic Pressure Cell)
Core
Station 102+54
Elevation, Ft. MSL
5434
5432
5430
5428
5426
5424
5422
5420
5418
5416
28-Oct-95
11-Mar-97
24-Jul-98
6-Dec-99
19-Apr-01
1-Sep-02
14-Jan-04
28-May-05
10-Oct-06
22-Feb-08
Reservoir
519C
Tip El. = 5384.3

PLATE C-20

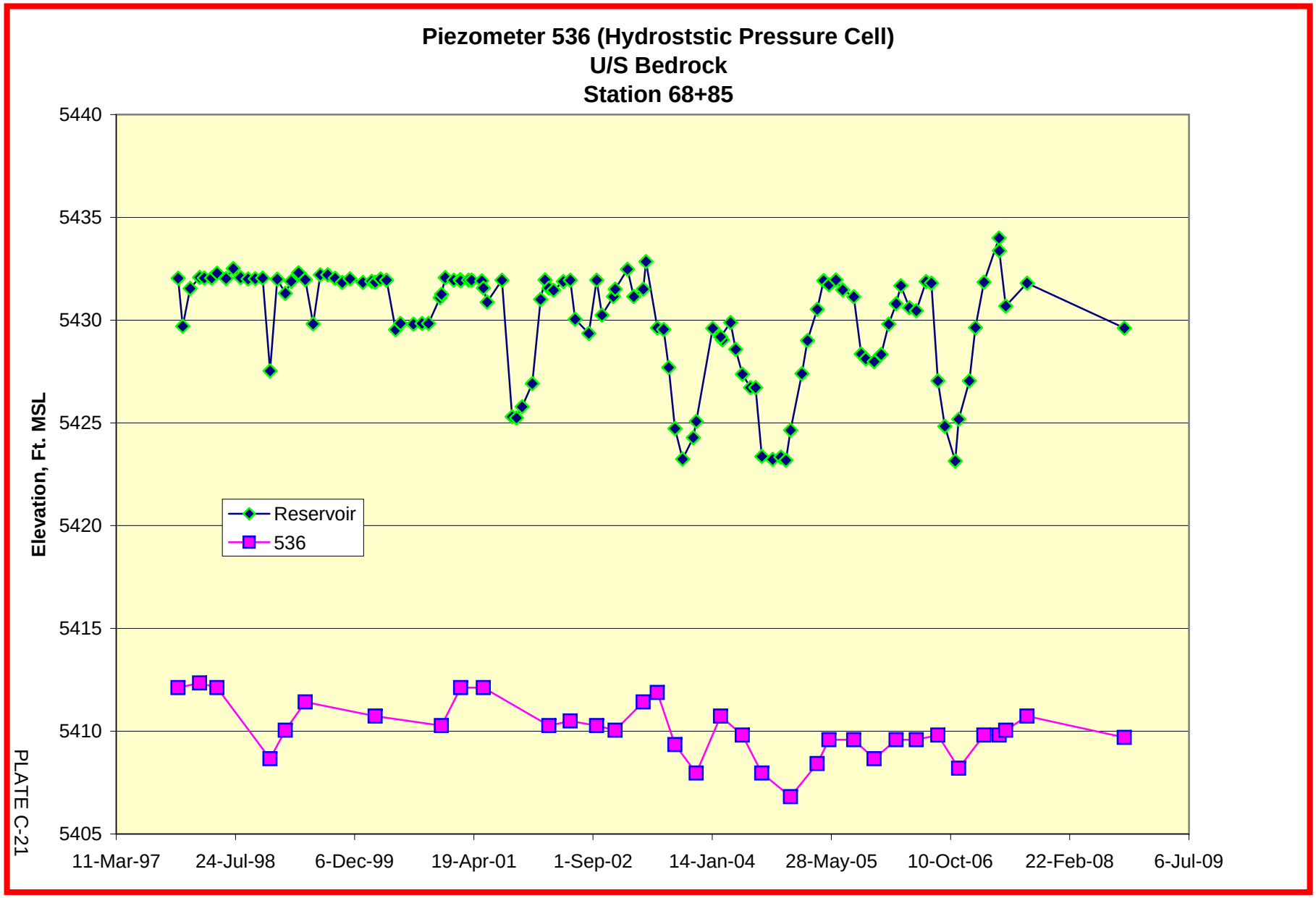
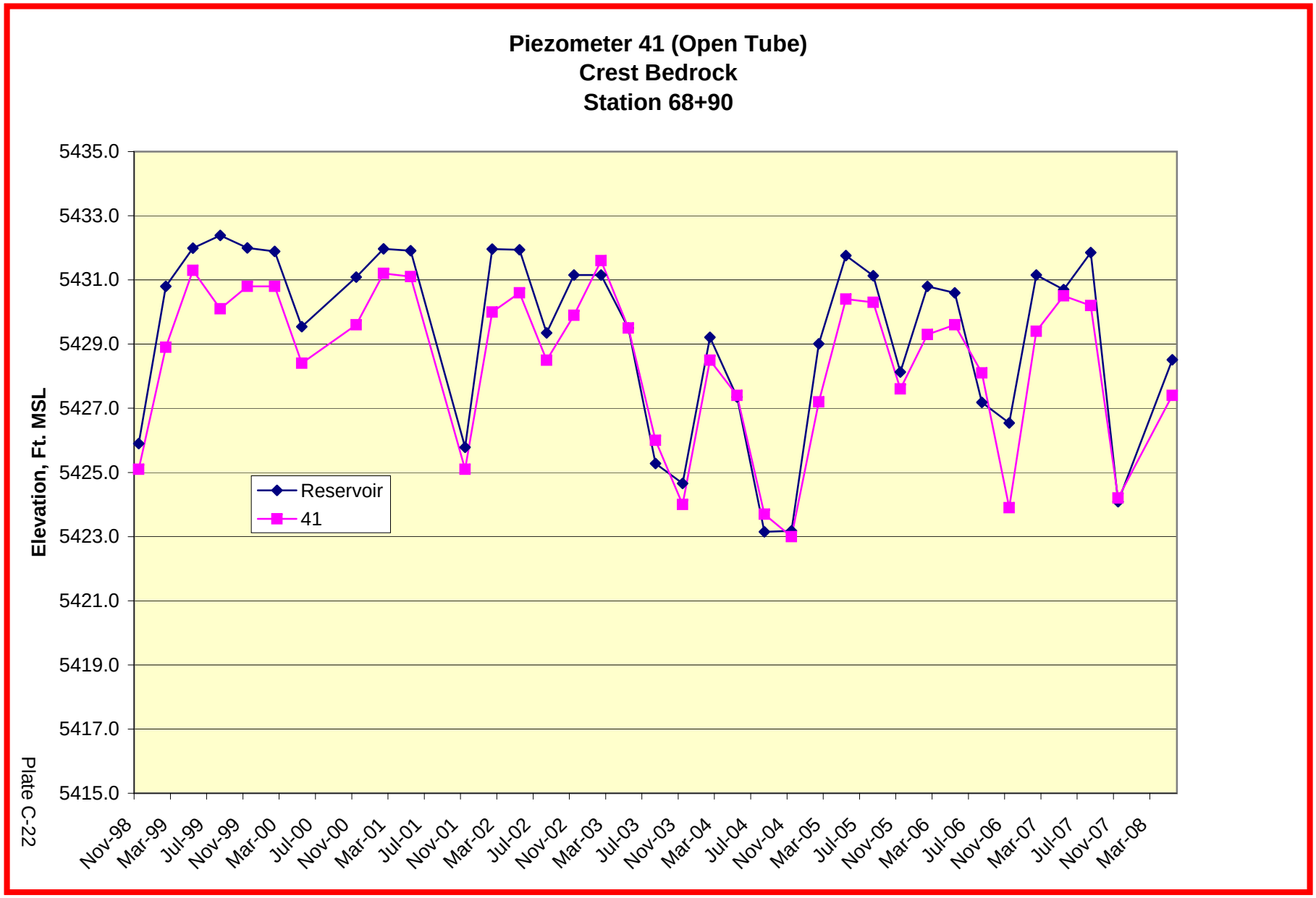

Piezometer 536 (Hydroststic Pressure Cell)
U/S Bedrock
Station 68+85
Elevation, Ft. MSL
5440
5435
5430
5425
5420
5415
5410
5405
11-Mar-97
24-Jul-98
6-Dec-99
19-Apr-01
1-Sep-02
14-Jan-04
28-May-05
10-Oct-06
22-Feb-08
6-Jul-09
Reservoir
536

PLATE C-21


Piezometer 41 (Open Tube)
Crest Bedrock
Station 68+90
Elevation, Ft. MSL
5435.0
5433.0
5431.0
5429.0
5427.0
5425.0
5423.0
5421.0
5419.0
5417.0
5415.0
Reservoir
41
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
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Mar-07
Jul-07
Nov-07
Mar-08

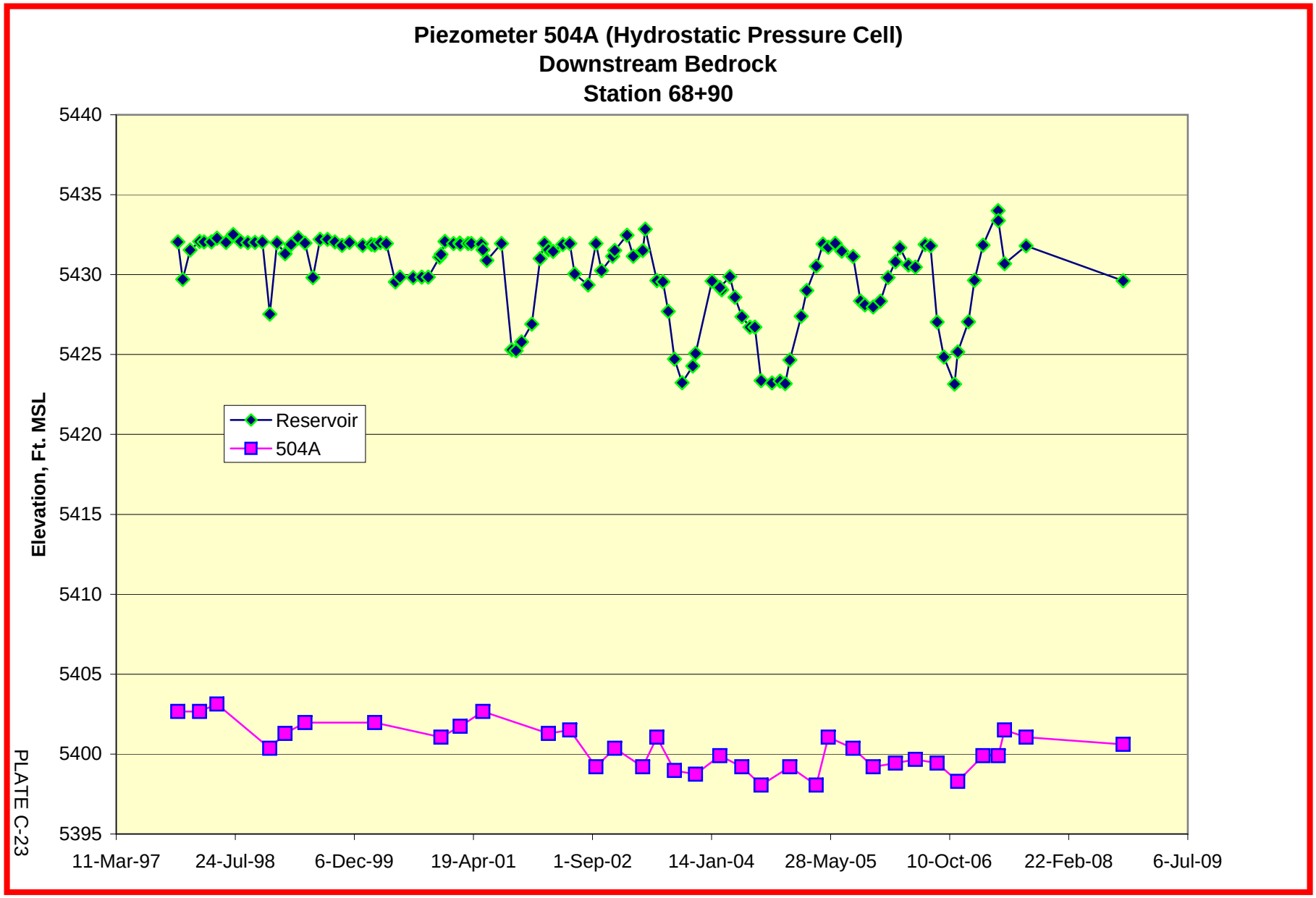

Piezometer 504A (Hydrostatic Pressure Cell)
Downstream Bedrock
Station 68+90
Elevation, Ft. MSL
5440
5435
5430
5425
5420
5415
5410
5405
5400
5395
Reservoir
504A
11-Mar-97
24-Jul-98
6-Dec-99
19-Apr-01
1-Sep-02
14-Jan-04
28-May-05
10-Oct-06
22-Feb-08
6-Jul-09

PLATE C-23

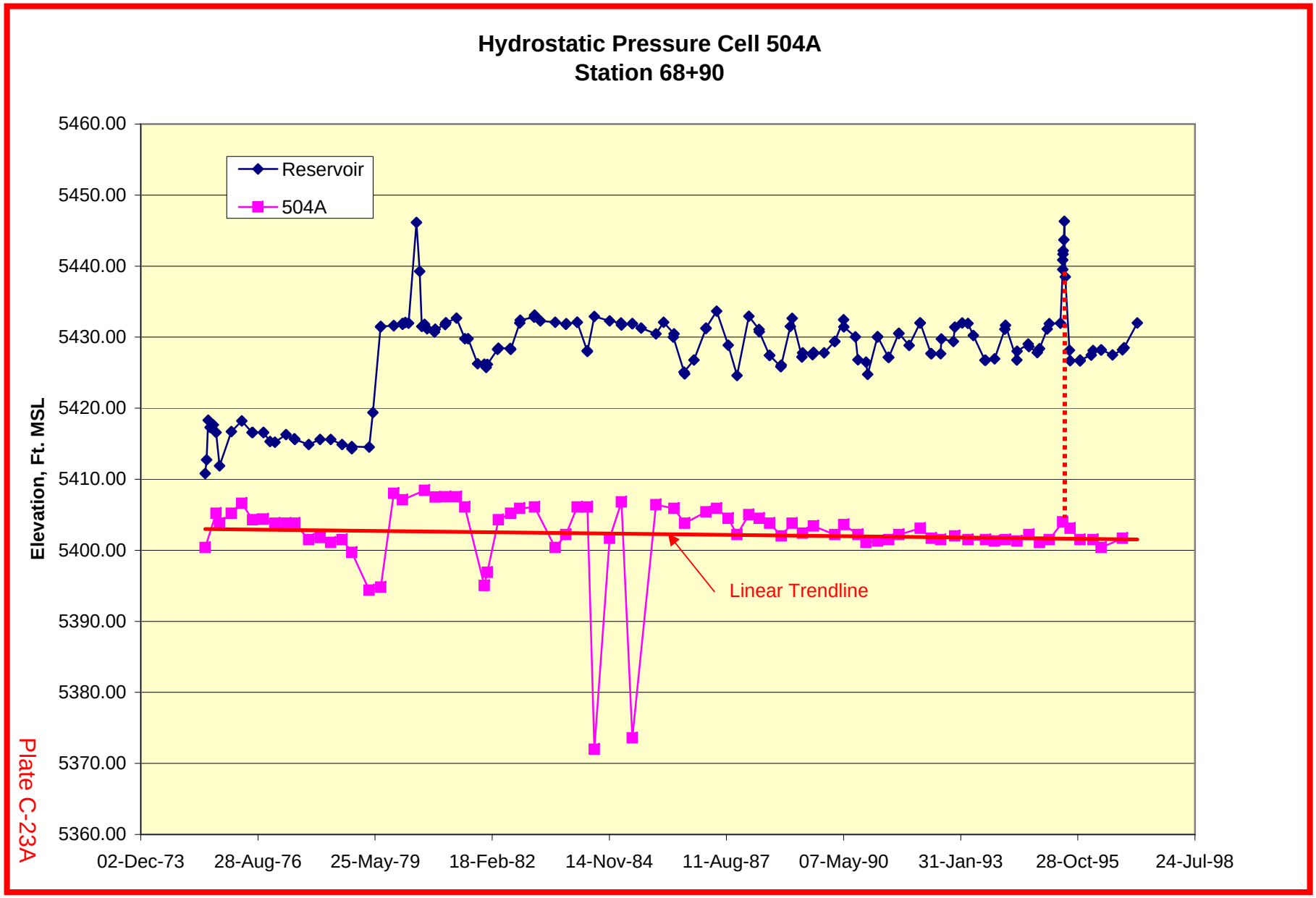

Hydrostatic Pressure Cell 504A
Station 68+90
Reservoir
504A
Linear Trendline
Elevation, Ft. MSL
5460.00
5450.00
5440.00
5430.00
5420.00
5410.00
5400.00
5390.00
5380.00
5370.00
5360.00
02-Dec-73
28-Aug-76
25-May-79
18-Feb-82
14-Nov-84
11-Aug-87
07-May-90
31-Jan-93
28-Oct-95
24-Jul-98

Plate C-23A

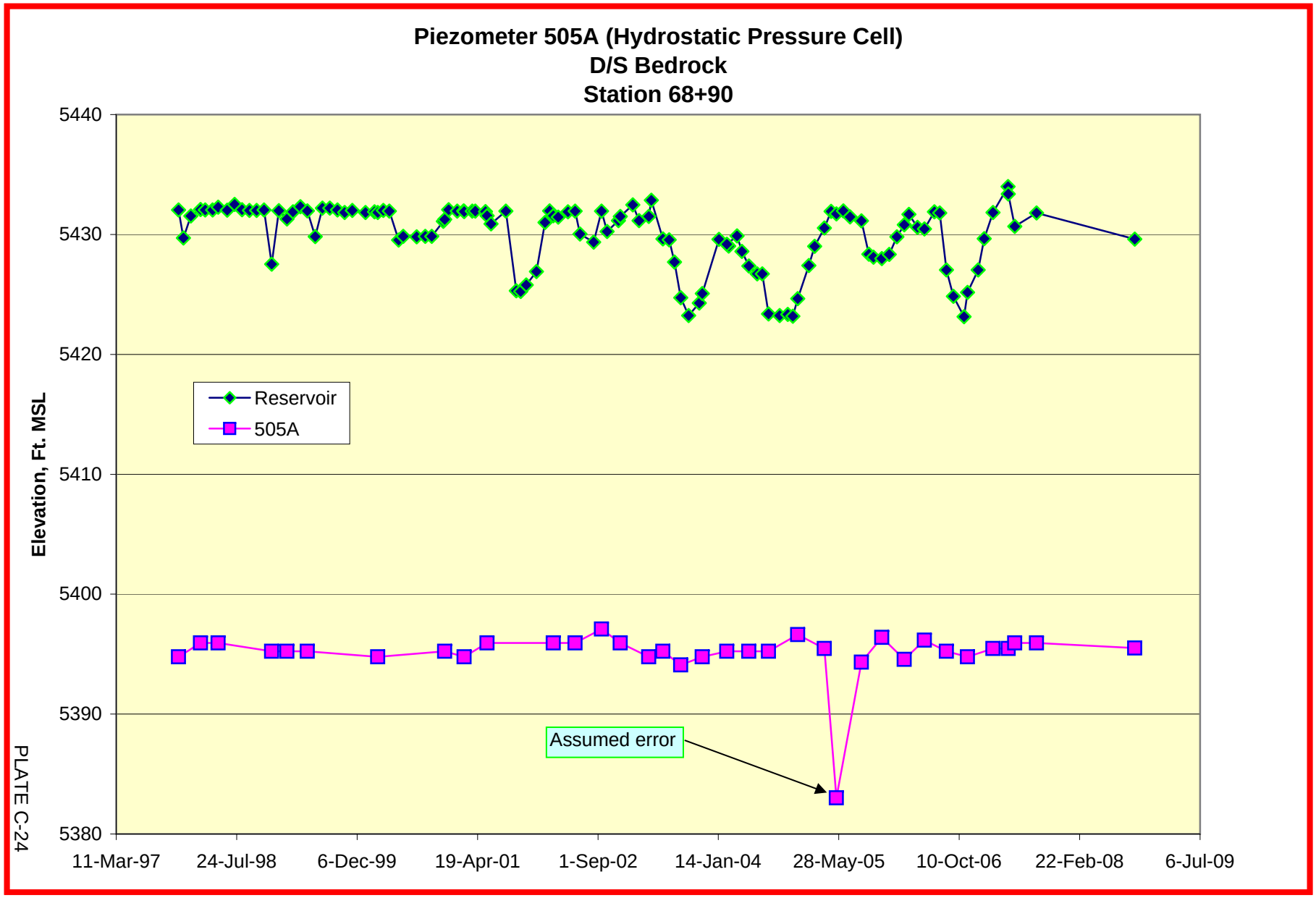
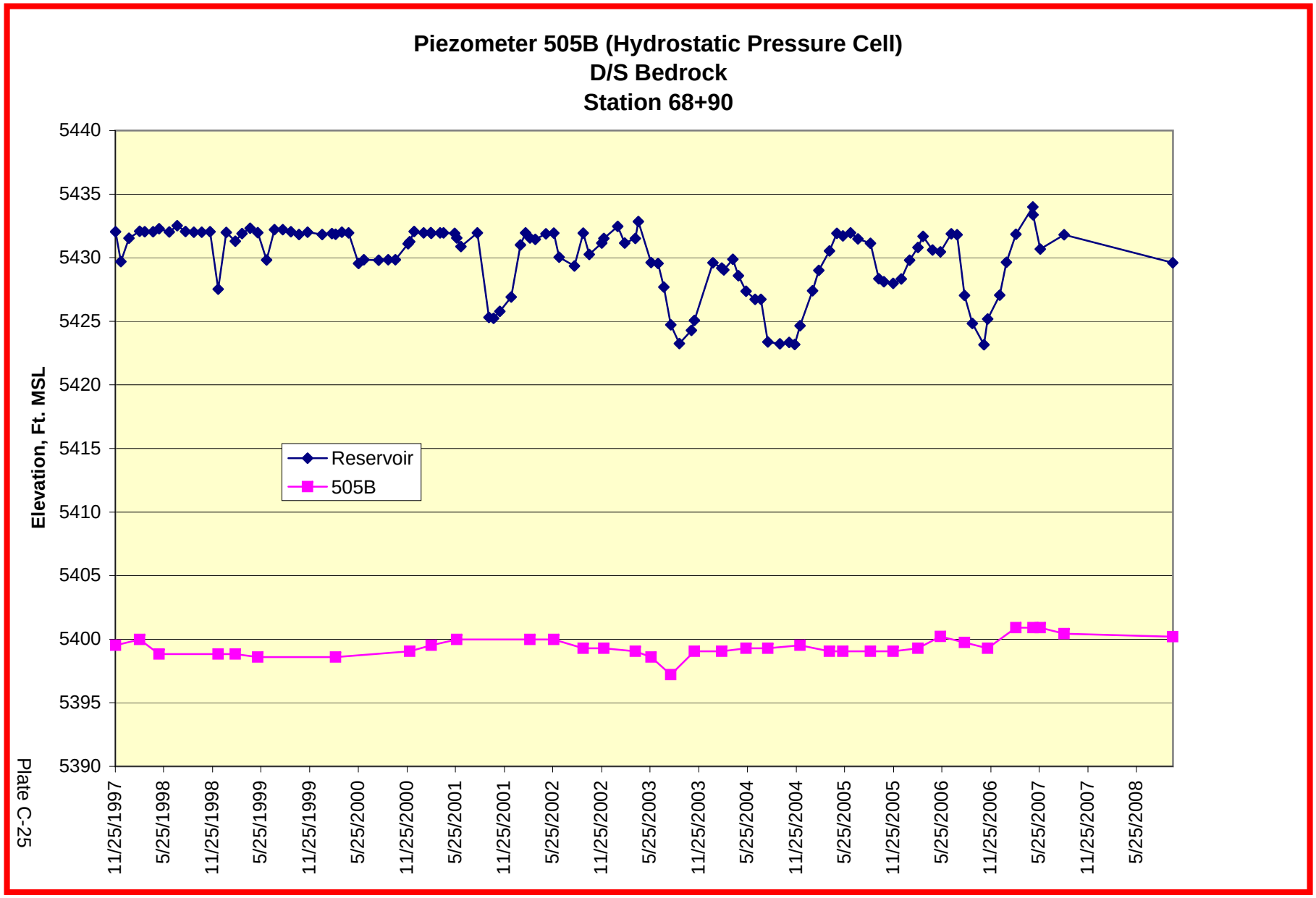

Piezometer 505A (Hydrostatic Pressure Cell)
D/S Bedrock
Station 68+90
Elevation, Ft. MSL
5440
5430
5420
5410
5400
5390
5380
Reservoir
505A
Assumed error
11-Mar-97
24-Jul-98
6-Dec-99
19-Apr-01
1-Sep-02
14-Jan-04
28-May-05
10-Oct-06
22-Feb-08
6-Jul-09

PLATE C-24


Piezometer 505B (Hydrostatic Pressure Cell)
D/S Bedrock
Station 68+90
Elevation, Ft. MSL
5440
5435
5430
5425
5420
5415
5410
5405
5400
5395
5390
Reservoir
505B
11/25/1997
5/25/1998
11/25/1998
5/25/1999
11/25/1999
5/25/2000
11/25/2000
5/25/2001
11/25/2001
5/25/2002
11/25/2002
5/25/2003
11/25/2003
5/25/2004
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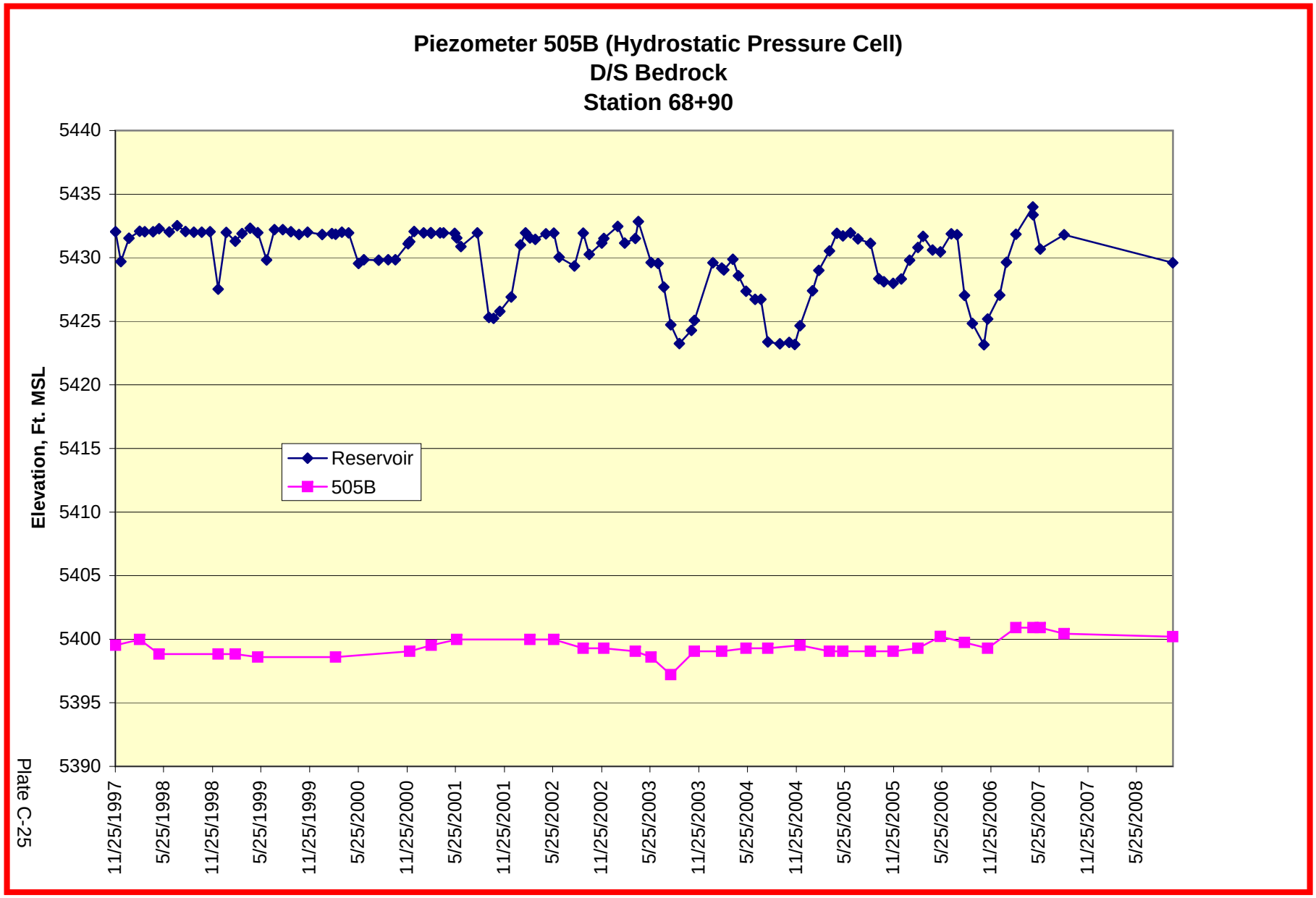
5/25/2007
11/25/2007
5/25/2008


Piezometer 505B (Hydrostatic Pressure Cell)
D/S Bedrock
Station 68+90
Elevation, Ft. MSL
5440
5435
5430
5425
5420
5415
5410
5405
5400
5395
5390
Reservoir
505B
11/25/1997
5/25/1998
11/25/1998
5/25/1999
11/25/1999
5/25/2000
11/25/2000
5/25/2001
11/25/2001
5/25/2002
11/25/2002
5/25/2003
11/25/2003
5/25/2004
11/25/2004
5/25/2005
11/25/2005
5/25/2006
11/25/2006
5/25/2007
11/25/2007
5/25/2008

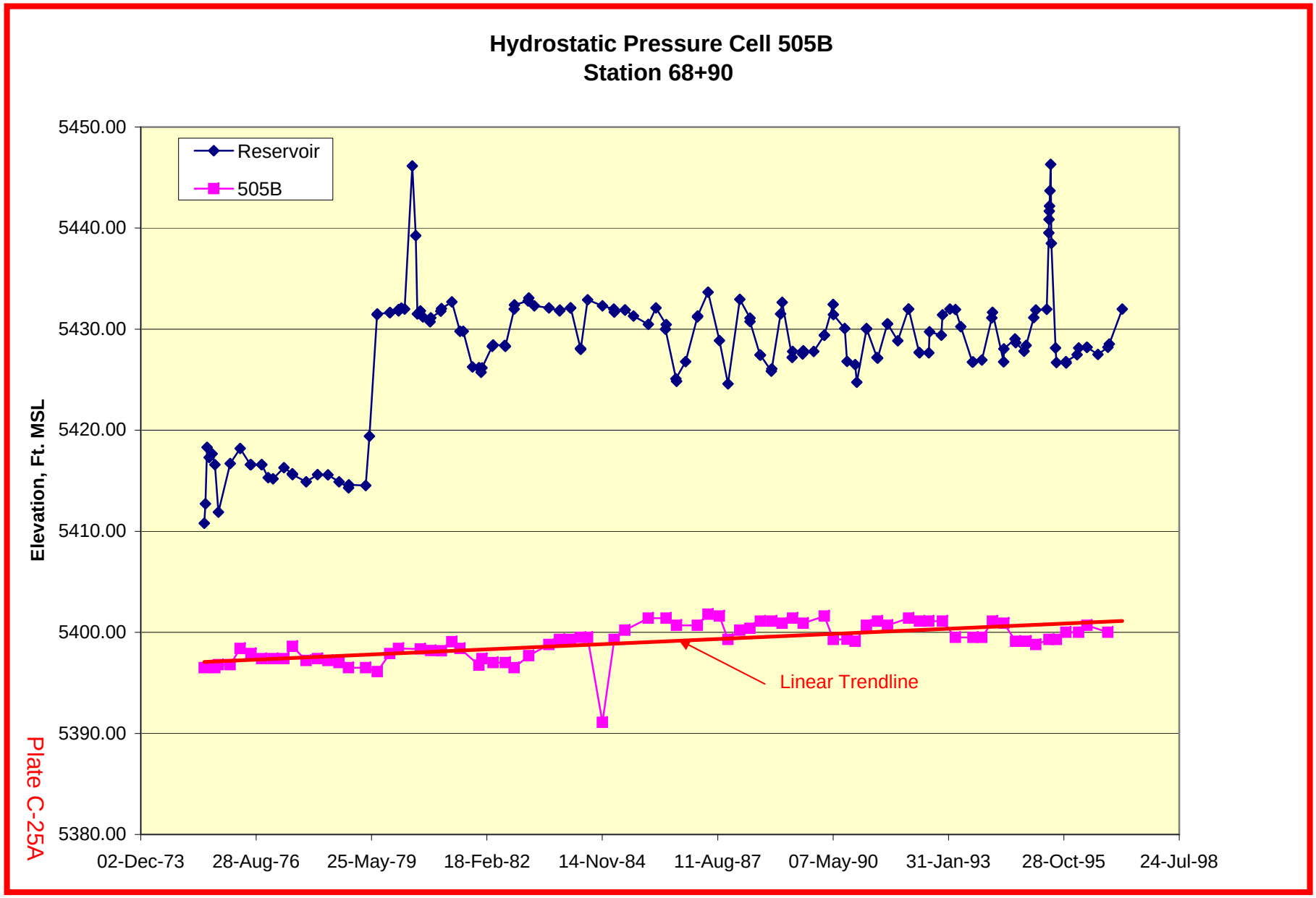
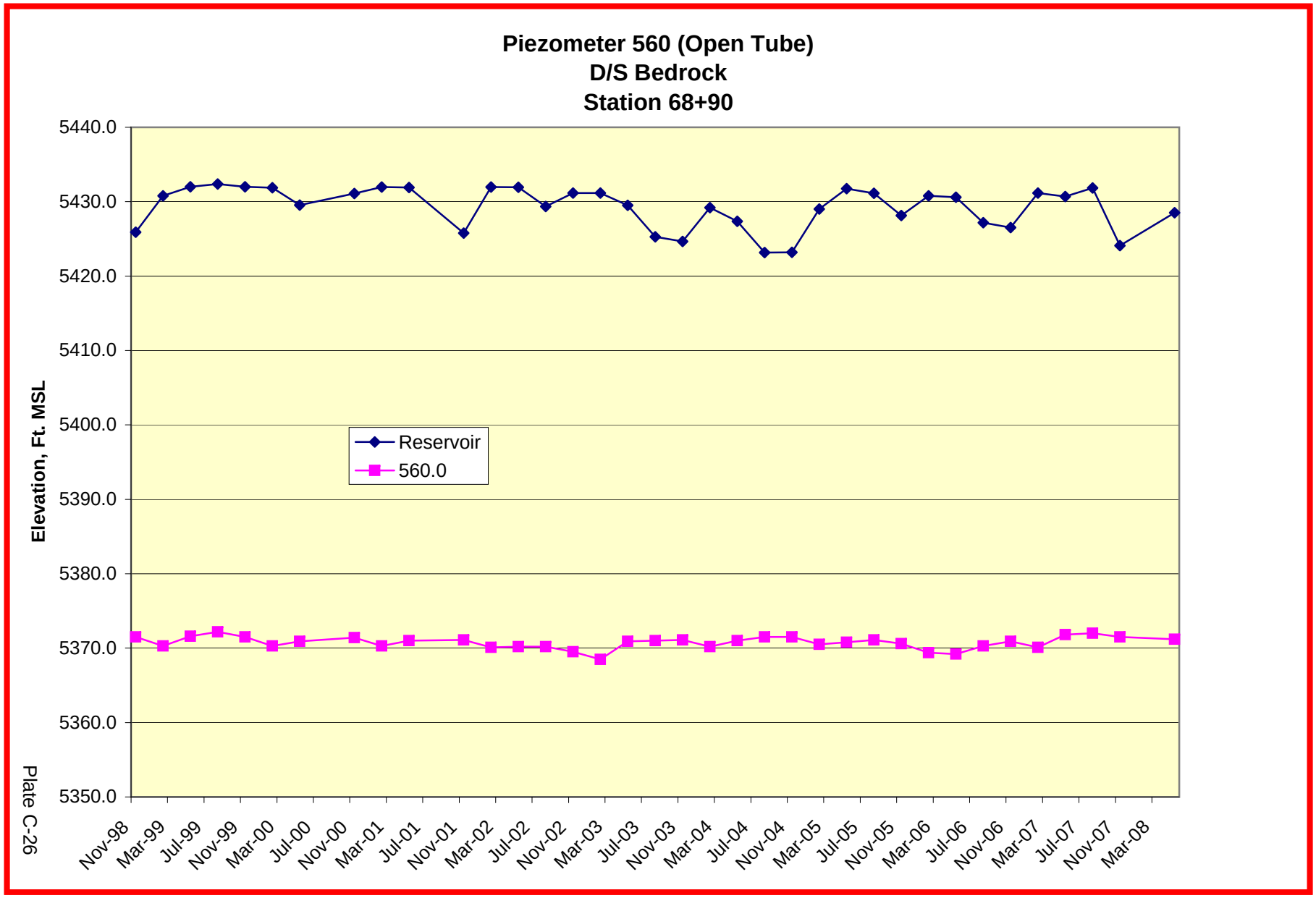

Hydrostatic Pressure Cell 505B
Station 68+90
Reservoir
505B
Linear Trendline
Elevation, Ft. MSL
5450.00
5440.00
5430.00
5420.00
5410.00
5400.00
5390.00
5380.00
02-Dec-73
28-Aug-76
25-May-79
18-Feb-82
14-Nov-84
11-Aug-87
07-May-90
31-Jan-93
28-Oct-95
24-Jul-98

Plate C-25A


Piezometer 560 (Open Tube)
D/S Bedrock
Station 68+90
Elevation, Ft. MSL
5440.0
5430.0
5420.0
5410.0
5400.0
5390.0
5380.0
5370.0
5360.0
5350.0
Reservoir
560.0
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
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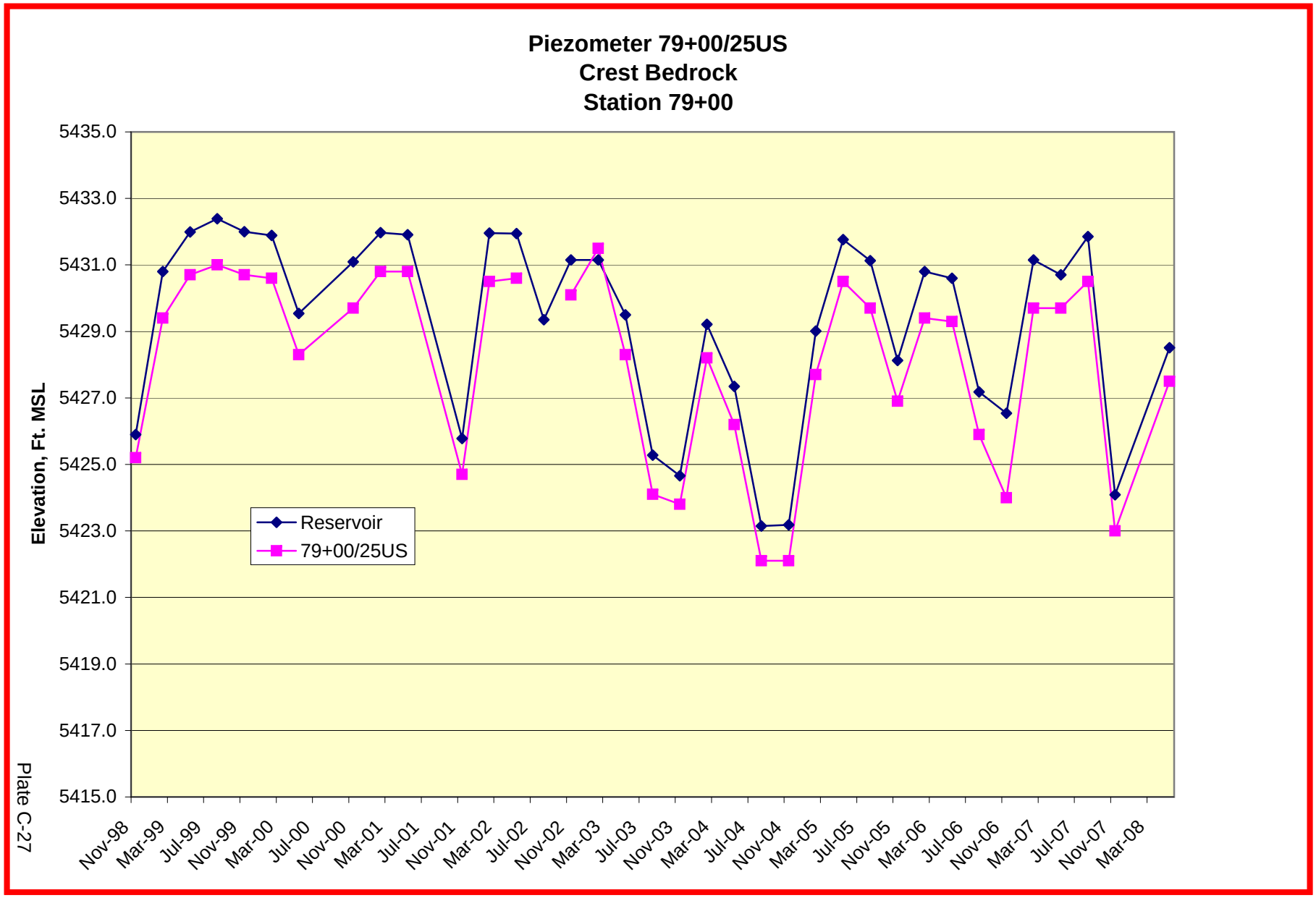
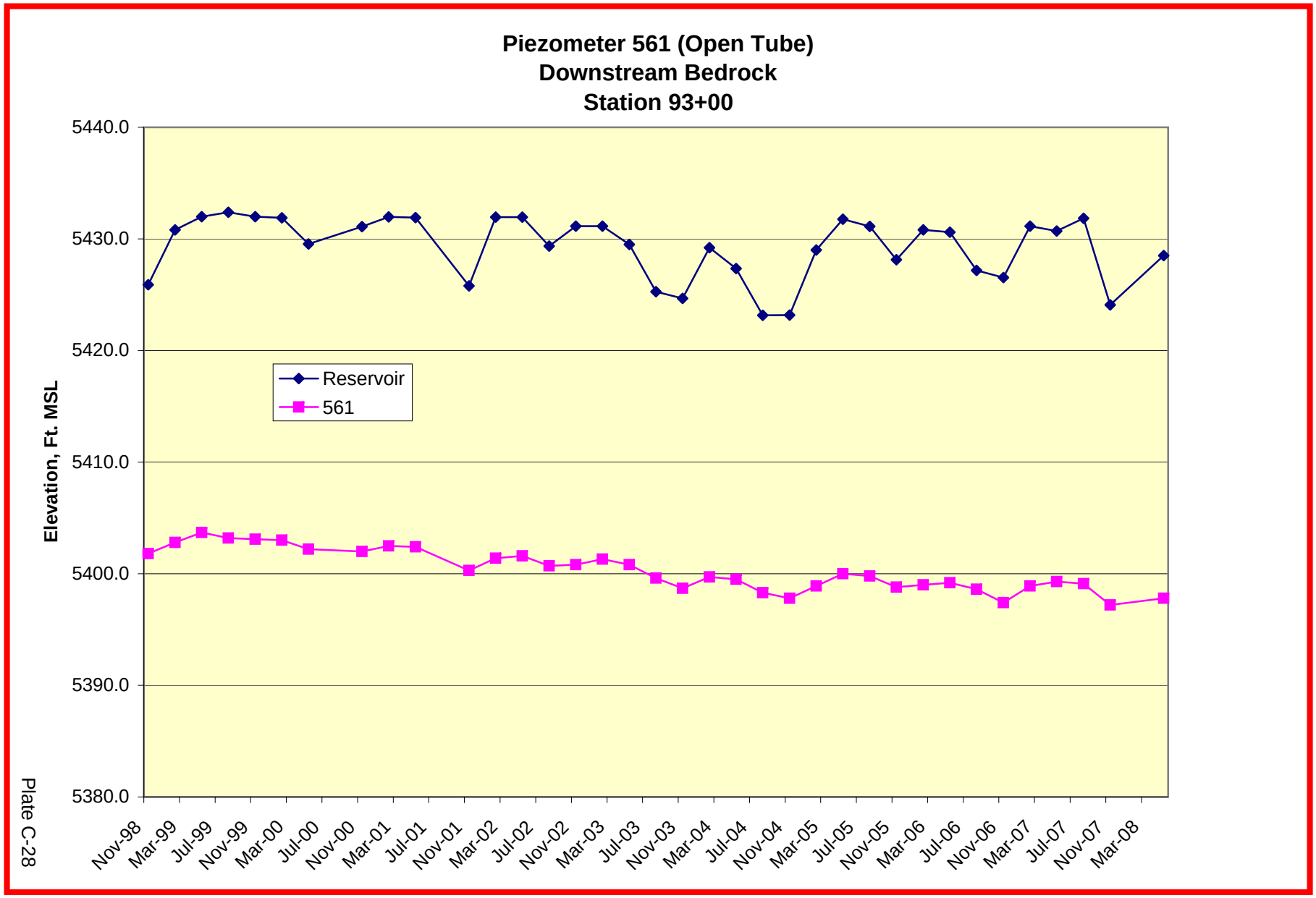
Mar-07
Jul-07
Nov-07
Mar-08


Piezometer 79+00/25US
Crest Bedrock
Station 79+00
Elevation, Ft. MSL
5435.0
5433.0
5431.0
5429.0
5427.0
5425.0
5423.0
5421.0
5419.0
5417.0
5415.0
Reservoir
79+00/25US
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
Jul-04
Nov-04
Mar-05
Jul-05
Nov-05
Mar-06
Jul-06
Nov-06
Mar-07
Jul-07
Nov-07
Mar-08


Piezometer 561 (Open Tube)
Downstream Bedrock
Station 93+00
Elevation, Ft. MSL
5440.0
5430.0
5420.0
5410.0
5400.0
5390.0
5380.0
Reservoir
561
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
Jul-04
Nov-04
Mar-05
Jul-05
Nov-05
Mar-06
Jul-06
Nov-06
Mar-07
Jul-07
Nov-07
Mar-08

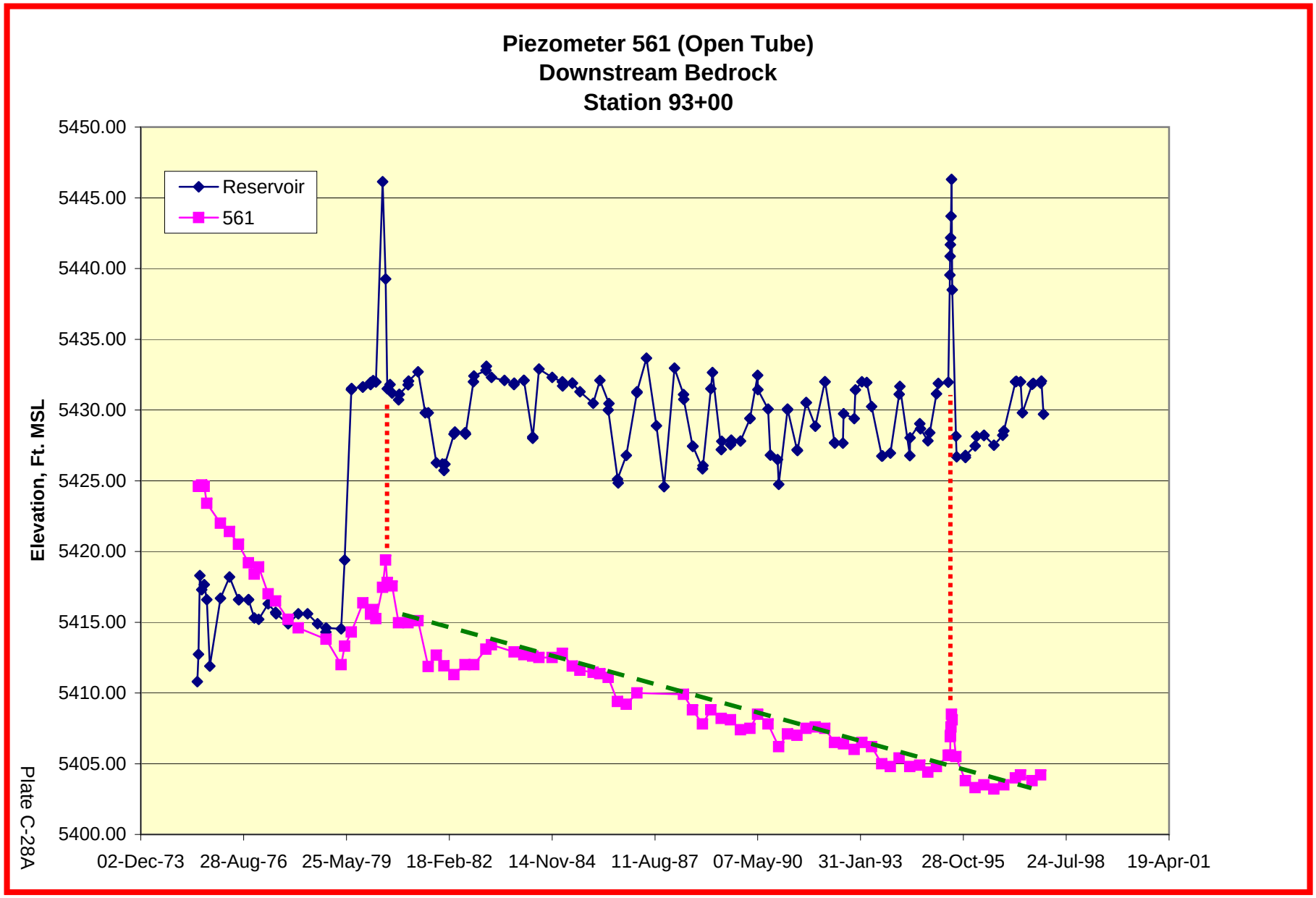
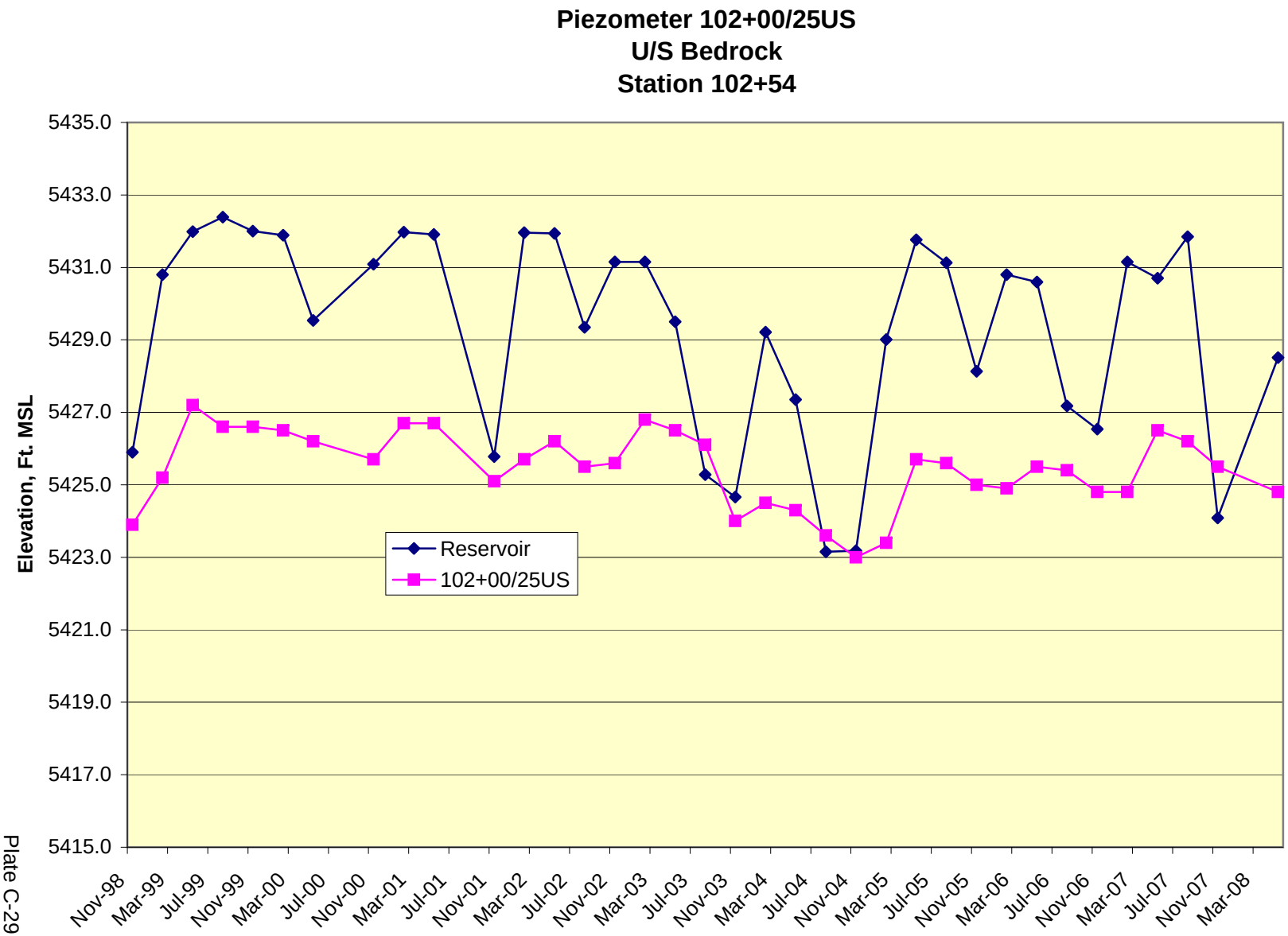

Piezometer 561 (Open Tube)
Downstream Bedrock
Station 93+00
Reservoir
561
Elevation, Ft. MSL
5450.00
5445.00
5440.00
5435.00
5430.00
5425.00
5420.00
5415.00
5410.00
5405.00
5400.00
02-Dec-73
28-Aug-76
25-May-79
18-Feb-82
14-Nov-84
11-Aug-87
07-May-90
31-Jan-93
28-Oct-95
24-Jul-98
19-Apr-01

Plate C-28A


Piezometer 102+00/25US
U/S Bedrock
Station 102+54
Elevation, Ft. MSL
5435.0
5433.0
5431.0
5429.0
5427.0
5425.0
5423.0
5421.0
5419.0
5417.0
5415.0
Reservoir
102+00/25US
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
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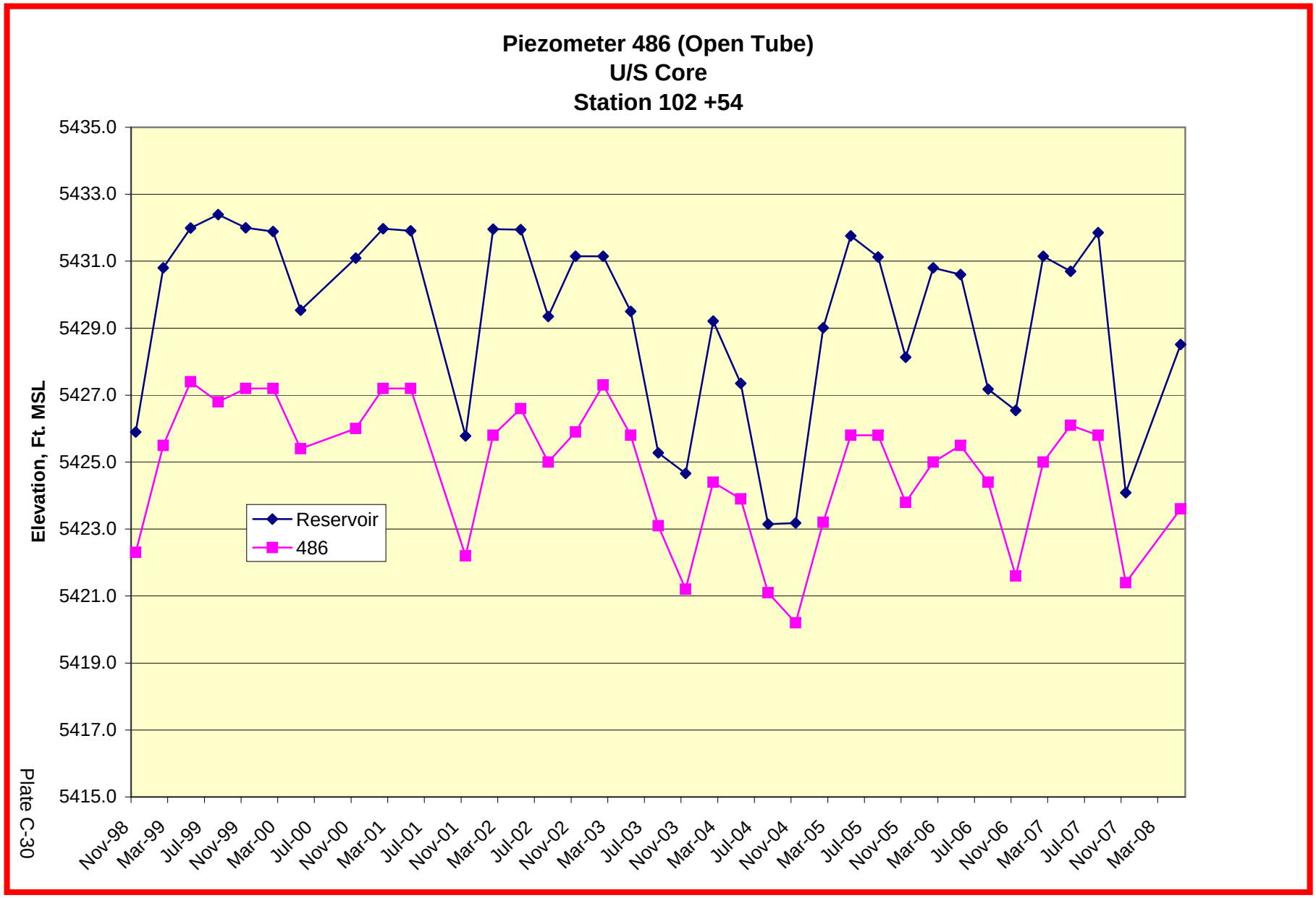
Mar-07
Jul-07
Nov-07
Mar-08


Piezometer 486 (Open Tube)
U/S Core
Station 102 +54
Elevation, Ft. MSL
5435.0
5433.0
5431.0
5429.0
5427.0
5425.0
5423.0
5421.0
5419.0
5417.0
5415.0
Reservoir
486
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
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Mar-07
Jul-07
Nov-07
Mar-08

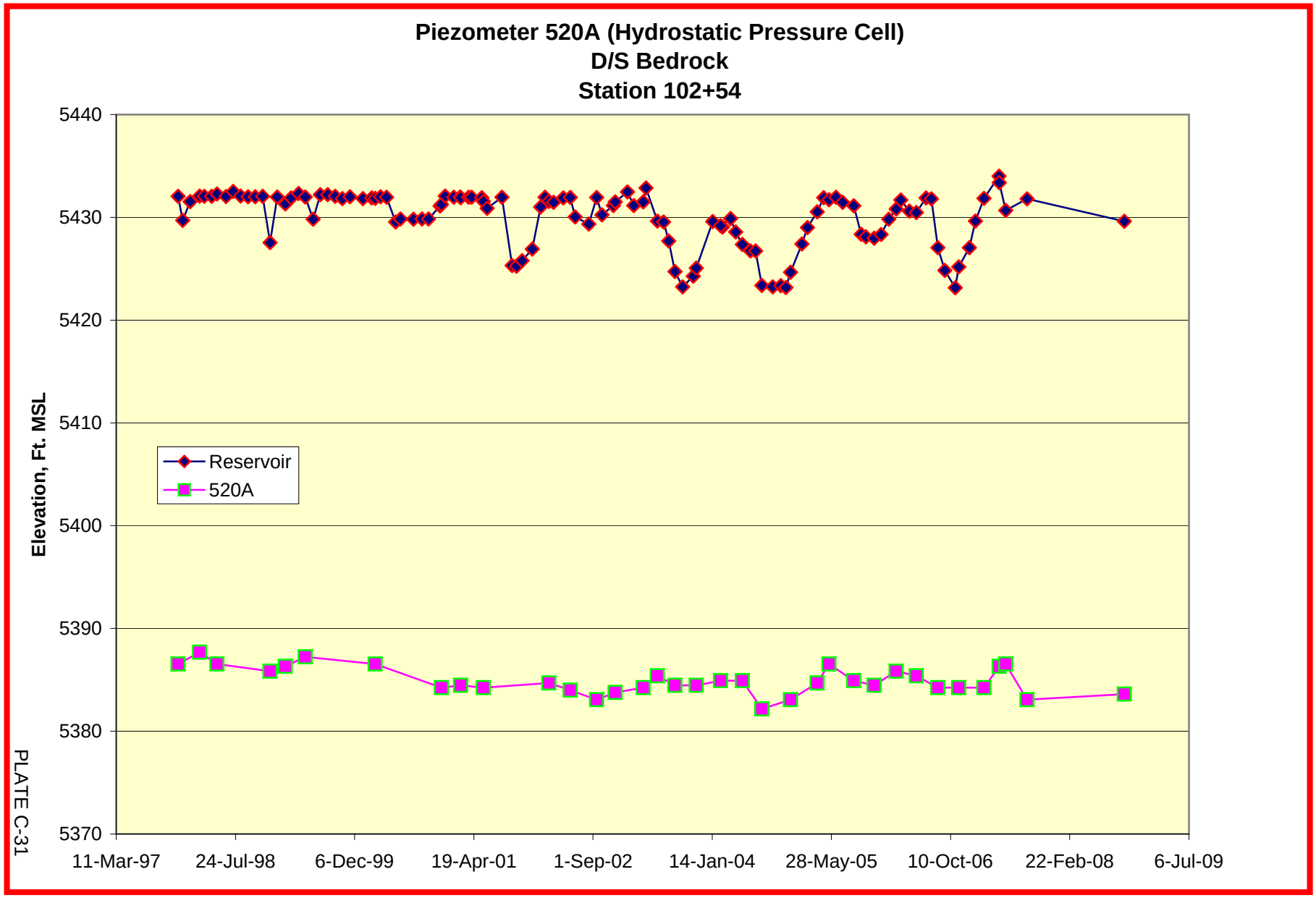

Piezometer 520A (Hydrostatic Pressure Cell)
D/S Bedrock
Station 102+54
Elevation, Ft. MSL
5440
5430
5420
5410
5400
5390
5380
5370
Reservoir
520A
11-Mar-97
24-Jul-98
6-Dec-99
19-Apr-01
1-Sep-02
14-Jan-04
28-May-05
10-Oct-06
22-Feb-08
6-Jul-09

PLATE C-31

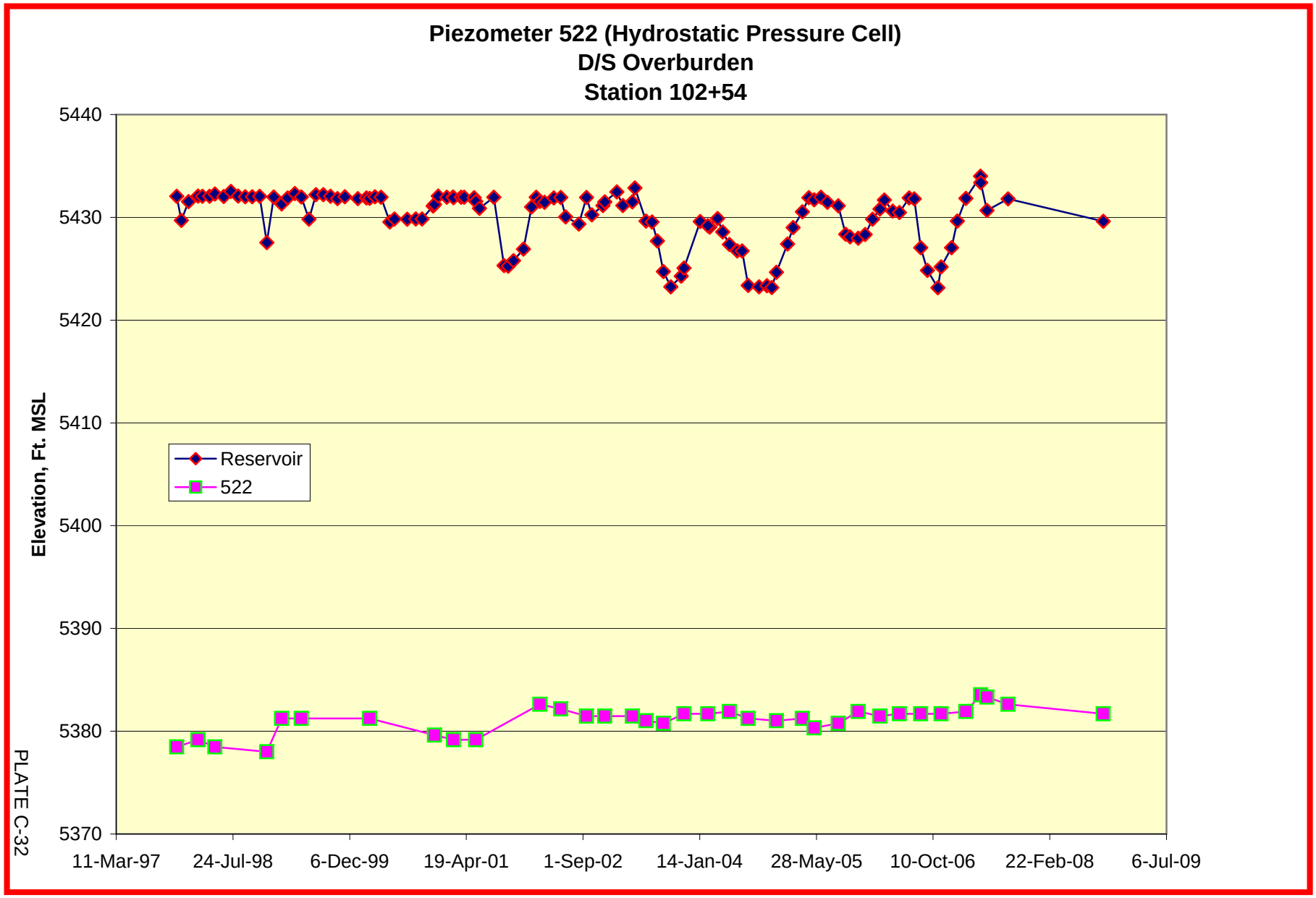
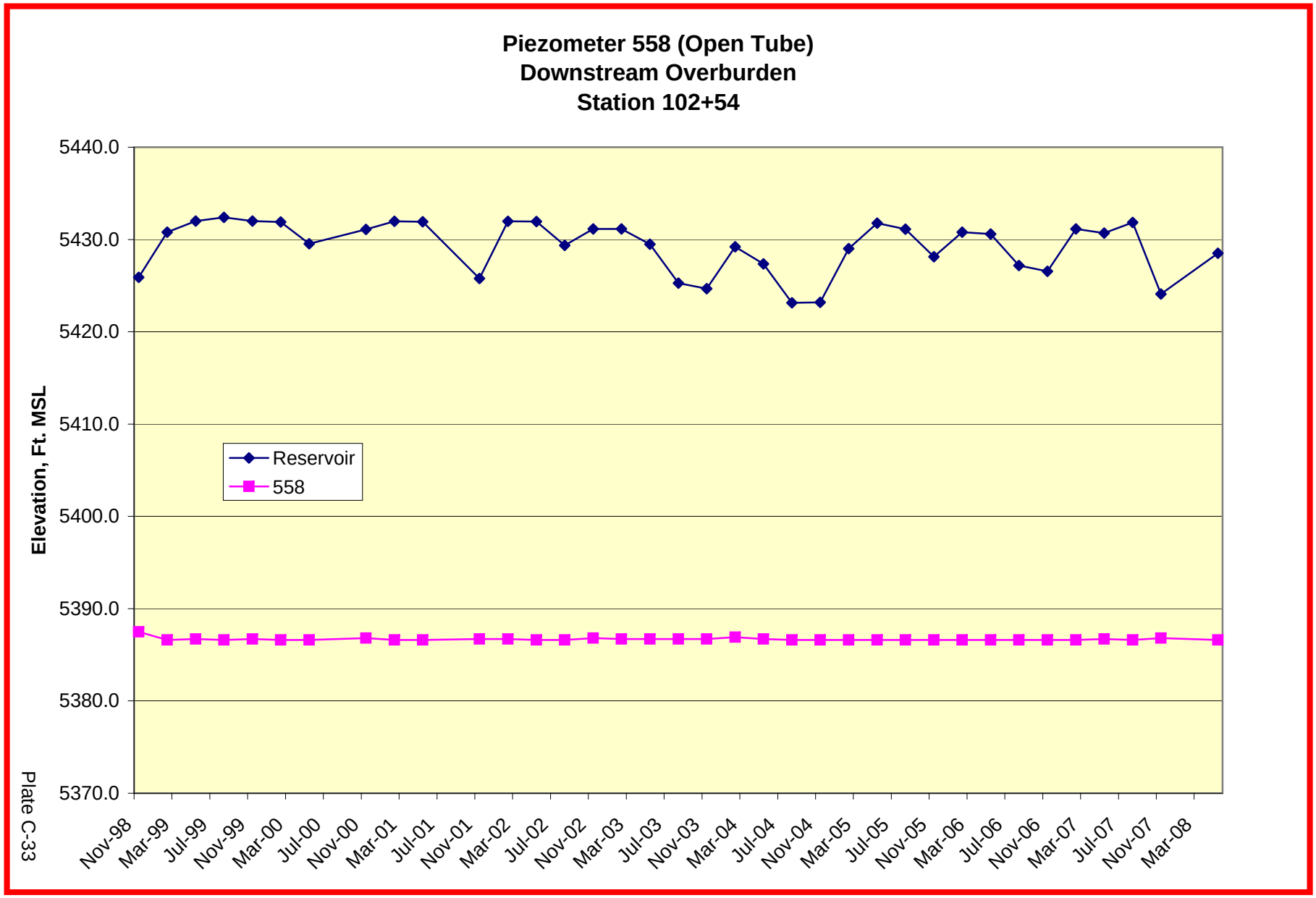
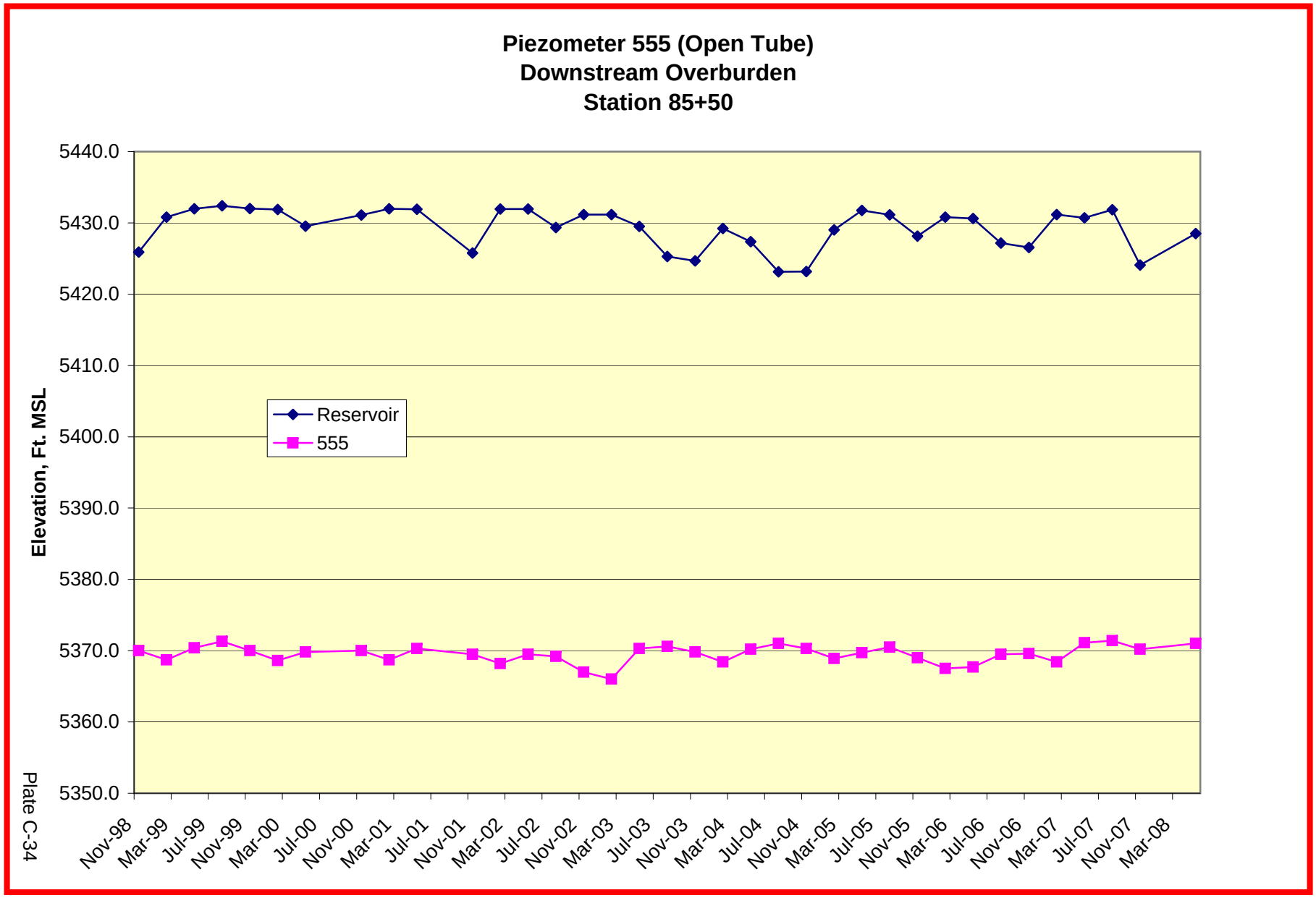
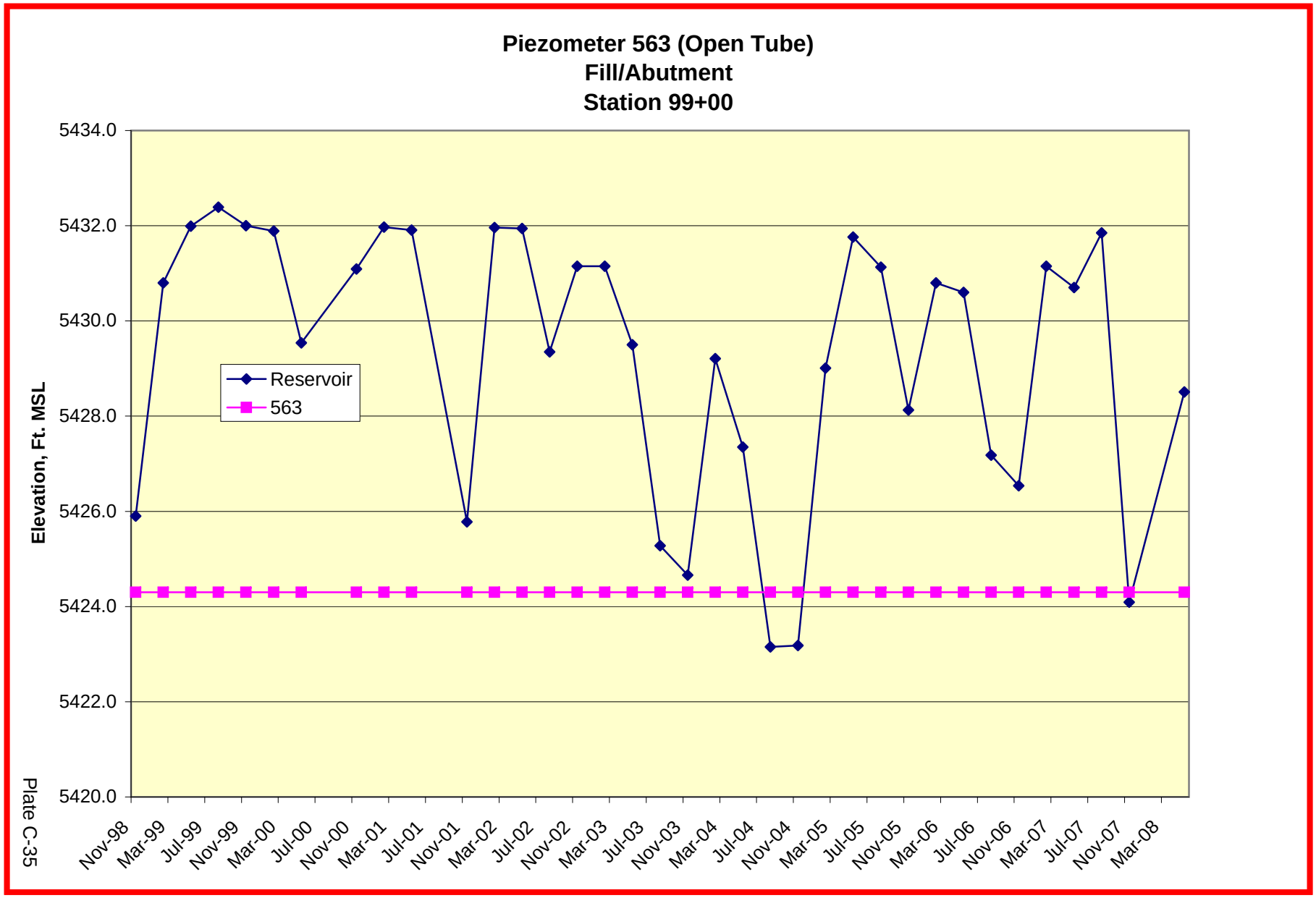

Piezometer 522 (Hydrostatic Pressure Cell)
D/S Overburden
Station 102+54
Elevation, Ft. MSL
5440
5430
5420
5410
5400
5390
5380
5370
Reservoir
522
11-Mar-97
24-Jul-98
6-Dec-99
19-Apr-01
1-Sep-02
14-Jan-04
28-May-05
10-Oct-06
22-Feb-08
6-Jul-09

PLATE C-32


Piezometer 558 (Open Tube)
Downstream Overburden
Station 102+54
Elevation, Ft. MSL
5440.0
5430.0
5420.0
5410.0
5400.0
5390.0
5380.0
5370.0
Reservoir
558
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
Jul-04
Nov-04
Mar-05
Jul-05
Nov-05
Mar-06
Jul-06
Nov-06
Mar-07
Jul-07
Nov-07
Mar-08


Piezometer 555 (Open Tube)
Downstream Overburden
Station 85+50
Elevation, Ft. MSL
5440.0
5430.0
5420.0
5410.0
5400.0
5390.0
5380.0
5370.0
5360.0
5350.0
Reservoir
555
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
Jul-04
Nov-04
Mar-05
Jul-05
Nov-05
Mar-06
Jul-06
Nov-06
Mar-07
Jul-07
Nov-07
Mar-08


Piezometer 563 (Open Tube)
Fill/Abutment
Station 99+00
Elevation, Ft. MSL
5434.0
5432.0
5430.0
5428.0
5426.0
5424.0
5422.0
5420.0
Reservoir
563
Nov-98
Mar-99
Jul-99
Nov-99
Mar-00
Jul-00
Nov-00
Mar-01
Jul-01
Nov-01
Mar-02
Jul-02
Nov-02
Mar-03
Jul-03
Nov-03
Mar-04
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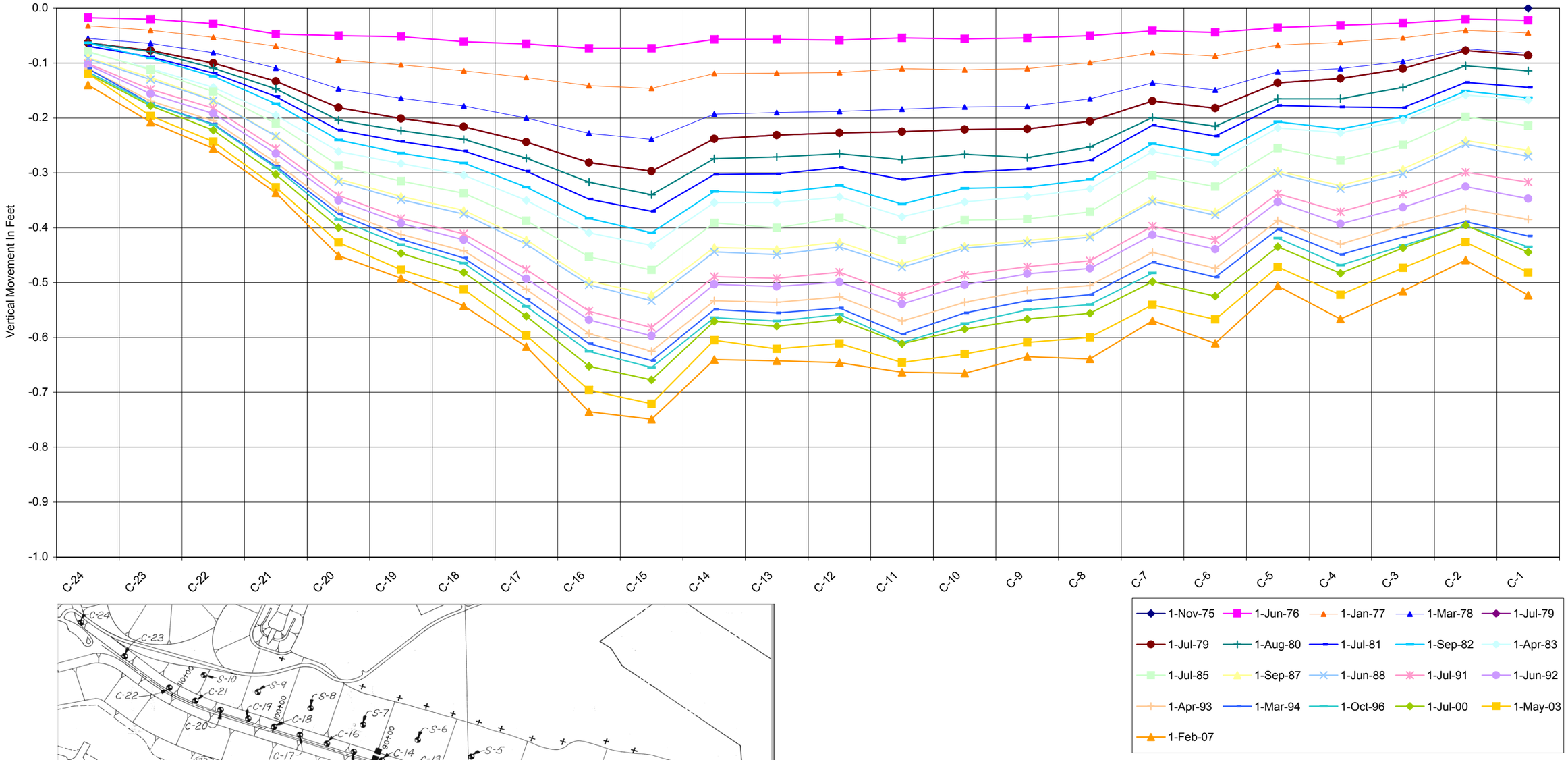
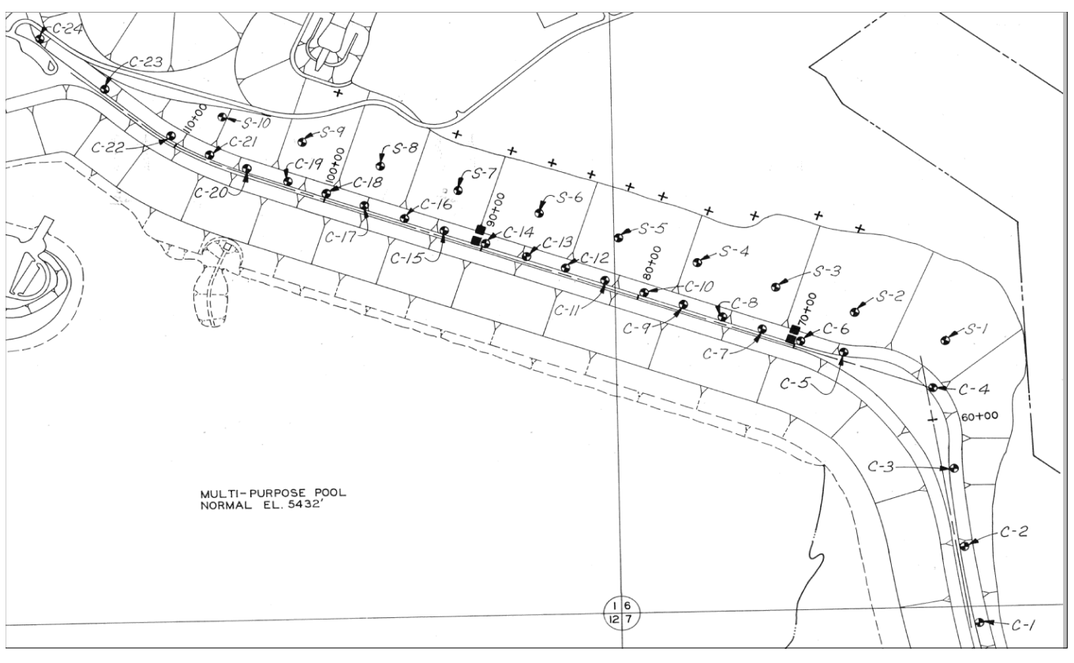
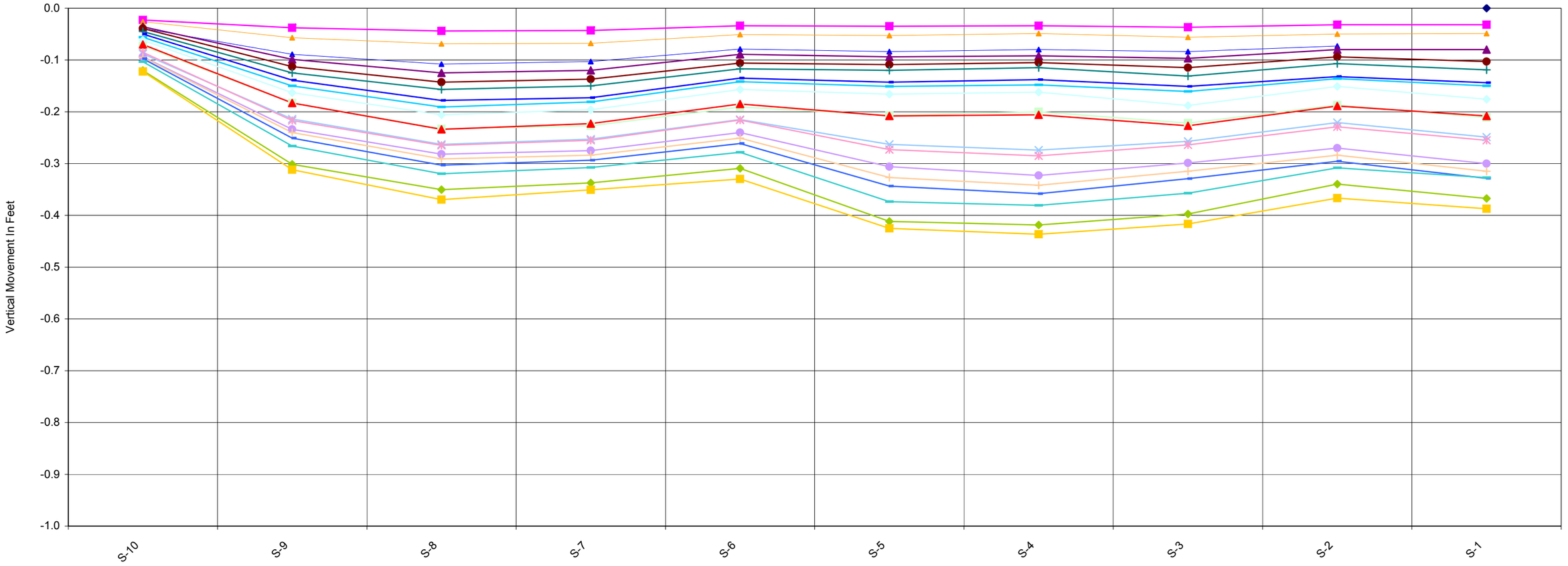
Mar-07
Jul-07
Nov-07
Mar-08

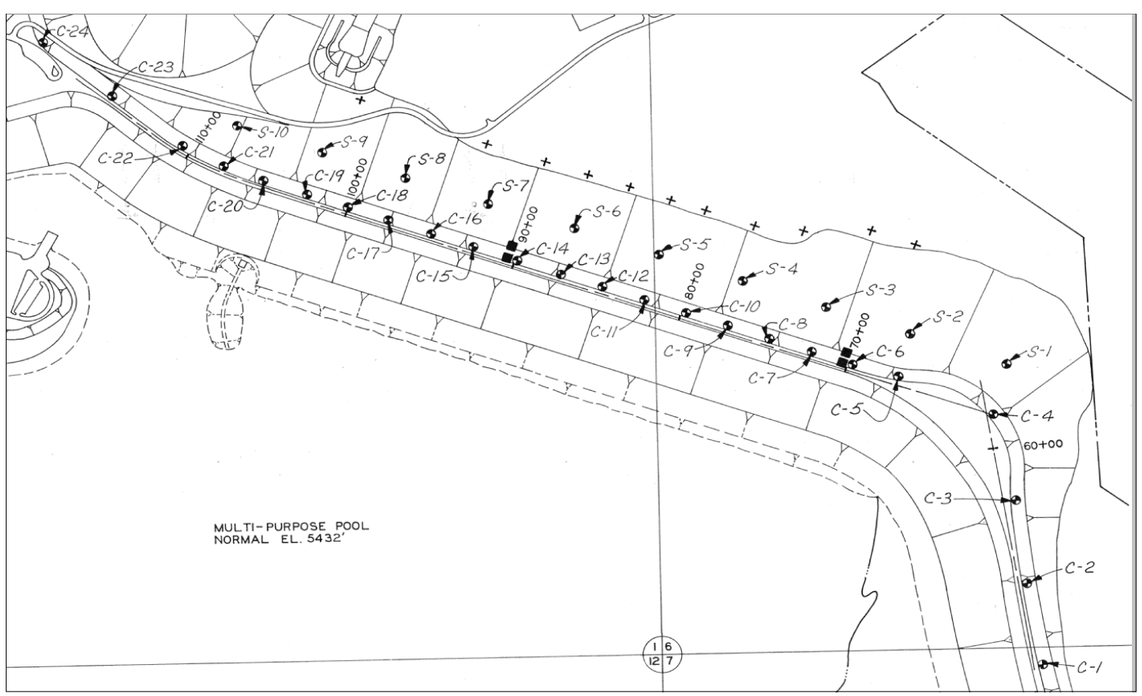




Chatfield Dam and Lake, CO
Embankment & Foundation Movement
Crest Movement Markers
Vertical Movement

Re-Allocation Study Plate D-1

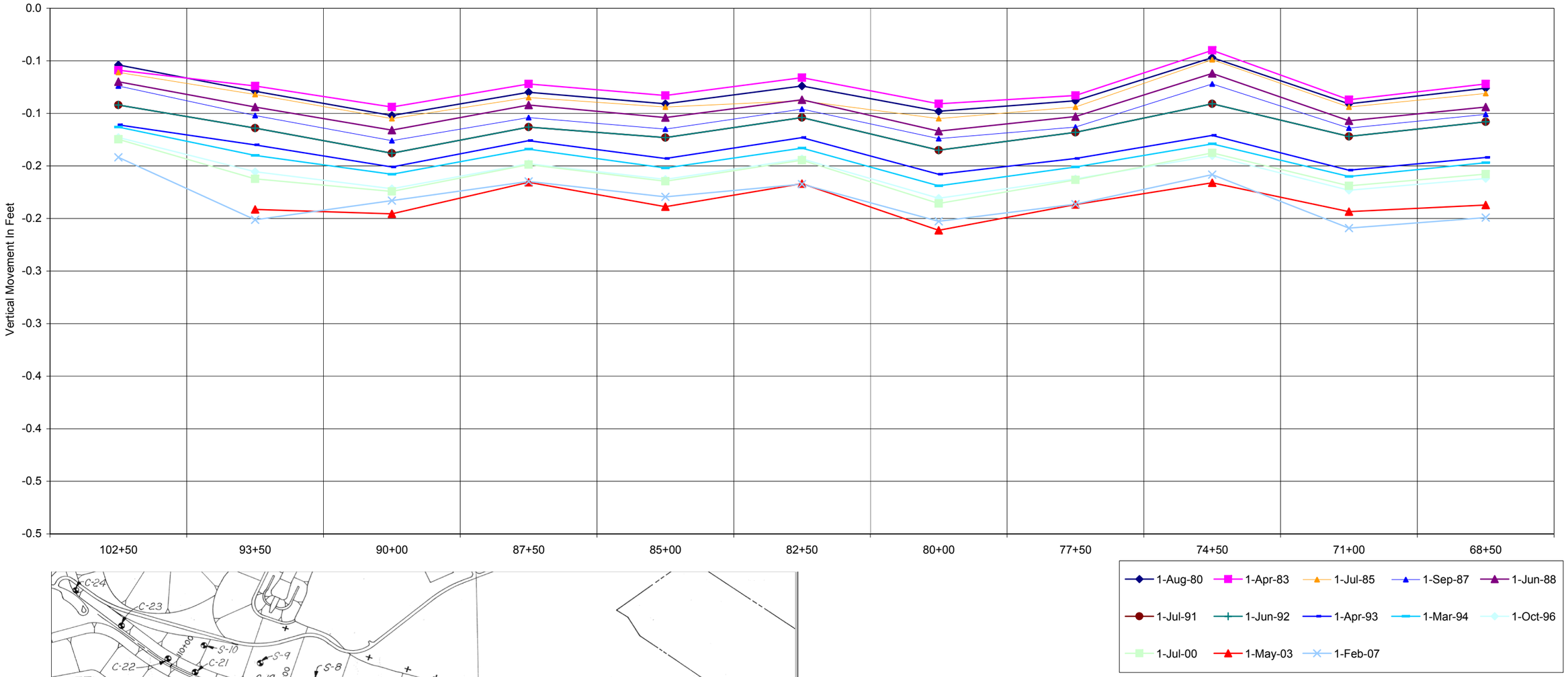


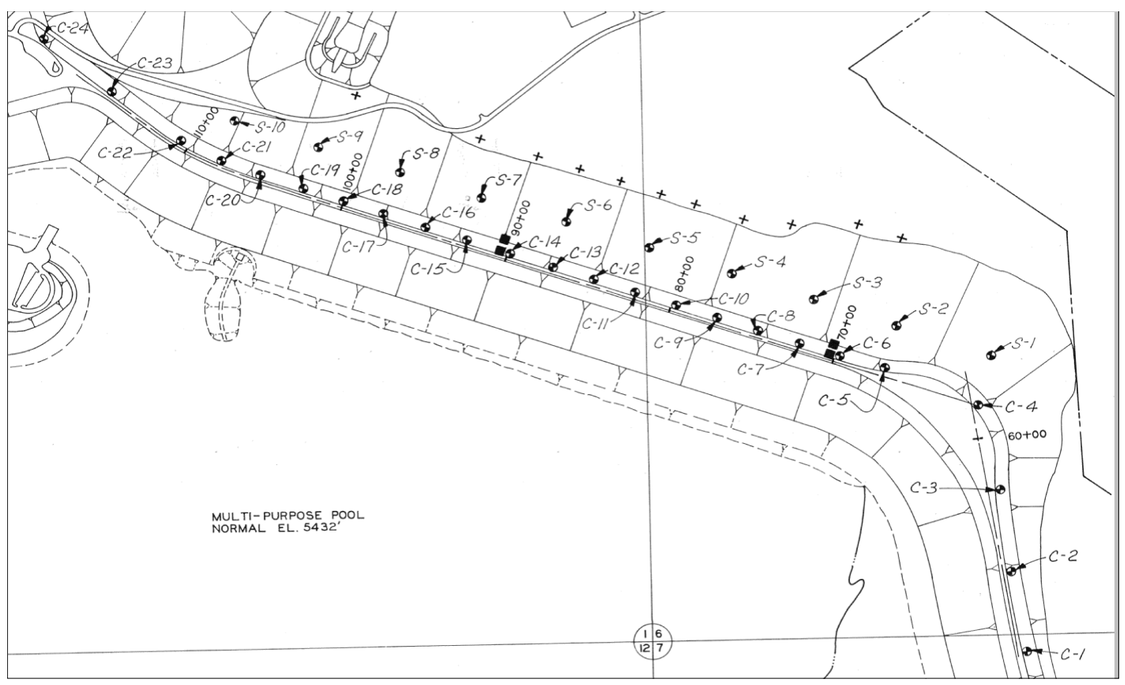


Notes:
1. Marker S-10 was found damaged and was reset August 1987.

Chatfield Dam and Lake, CO
Embankment & Foundation Movement
Slope Movement Markers
Vertical Movement

Re-Allocation Study Plate D-2

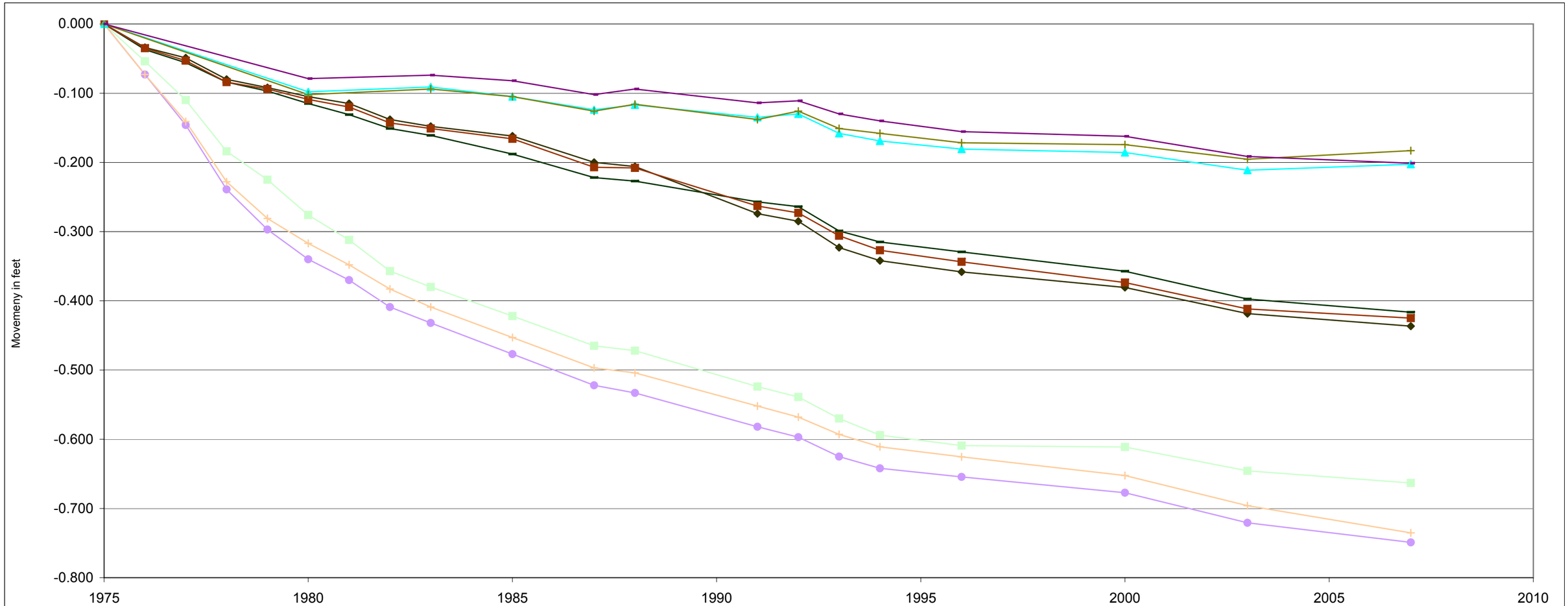


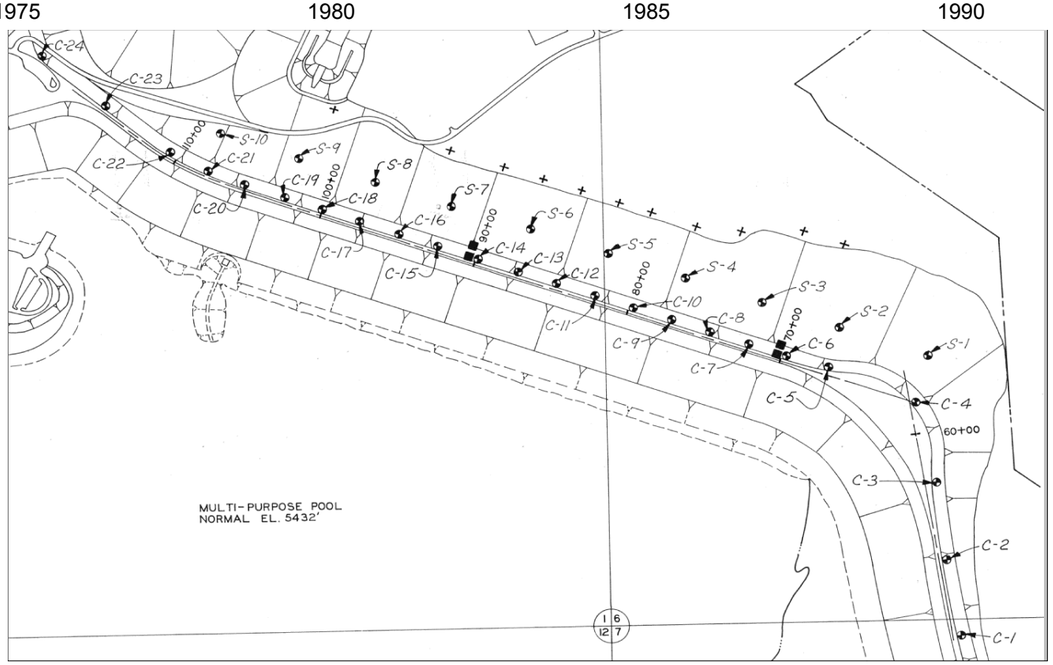


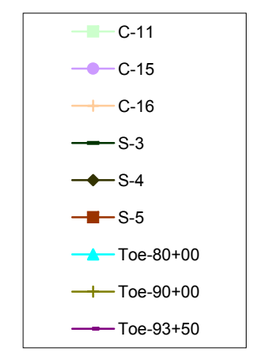
Notes:
1. 77+50 damaged and reset 8-24-71
2. 102+50 Initial 1974

Chatfield Dam and Lake, CO
Embankment & Foundation Movement
Toe Movement Markers
Vertical Movement

Re-Allocation Study Plate D-3

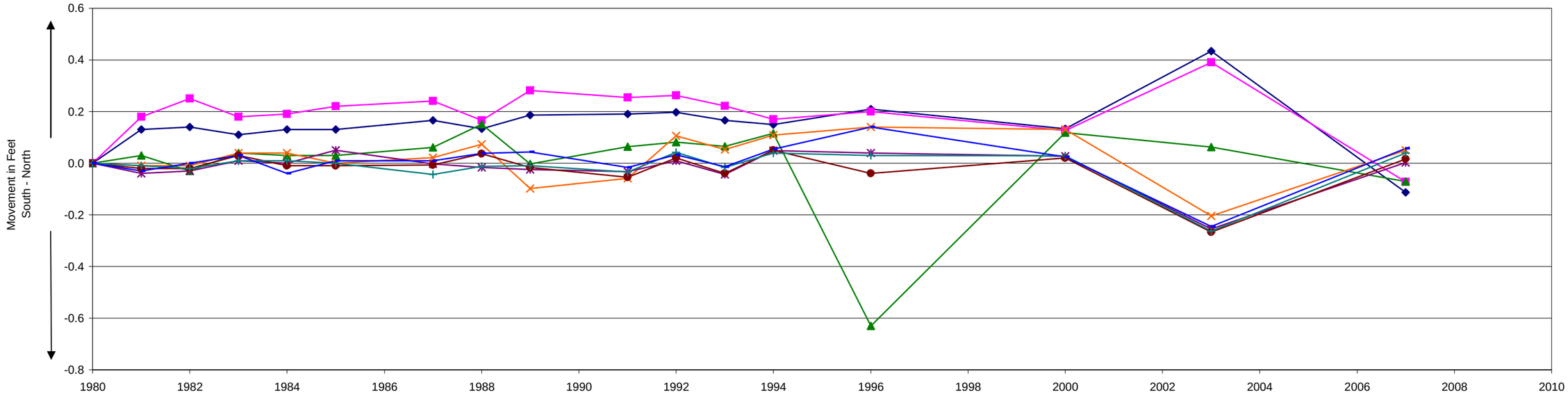


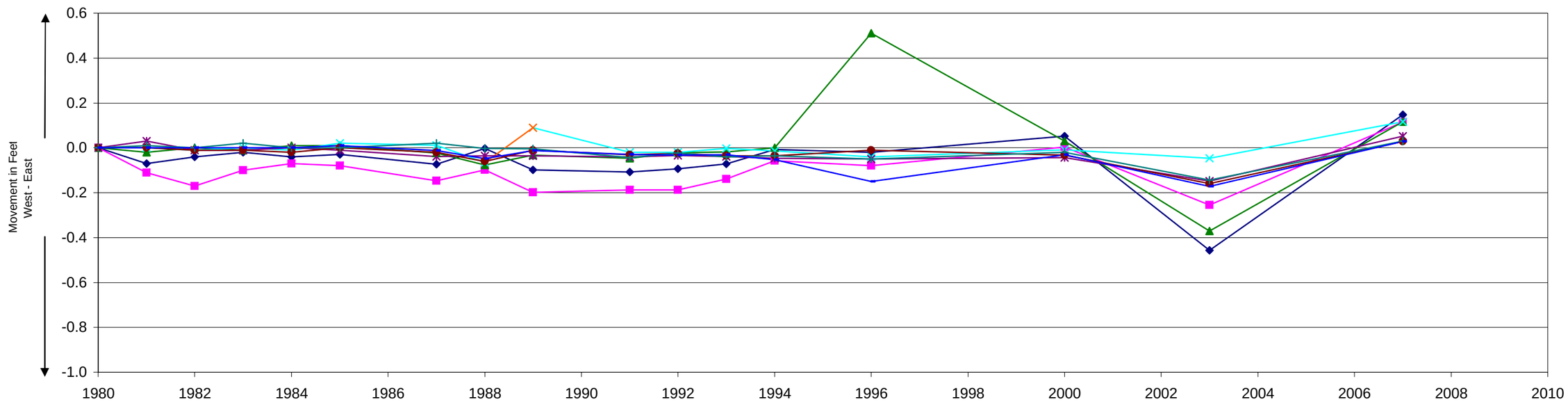




Chatfield Dam and Lake, CO
Embankment & Foundation Movement
Crest, Slope and Toe Movement Markers
Vertical Movement vs. Time Plot

Re-Allocation Study Plate D-4

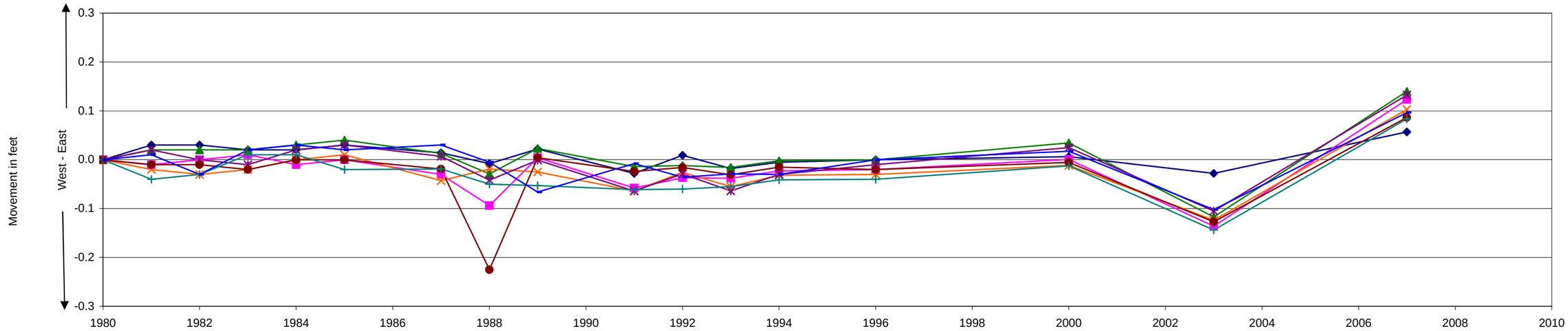


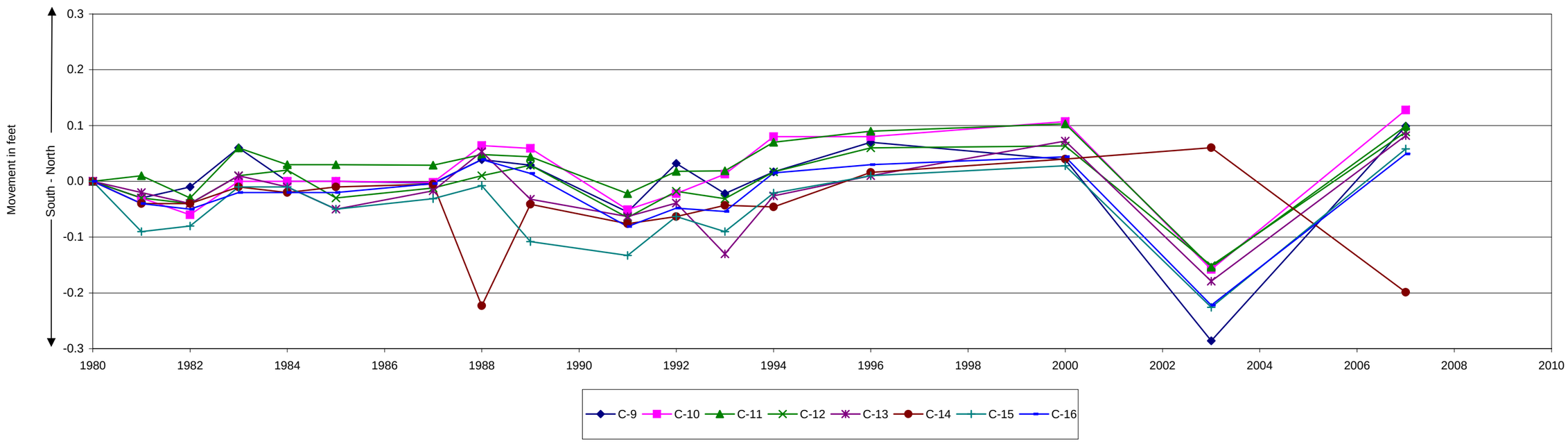


C-1 C-2 C-3 C-4 C-5 C-6 C-7 C-8

Chatfield Dam and Lake, CO
Embankment & Foundation Movement
Crest, Slope, and Toe Movement Markers
Horizontal Movement vs Time Plot Crest Markers 1-

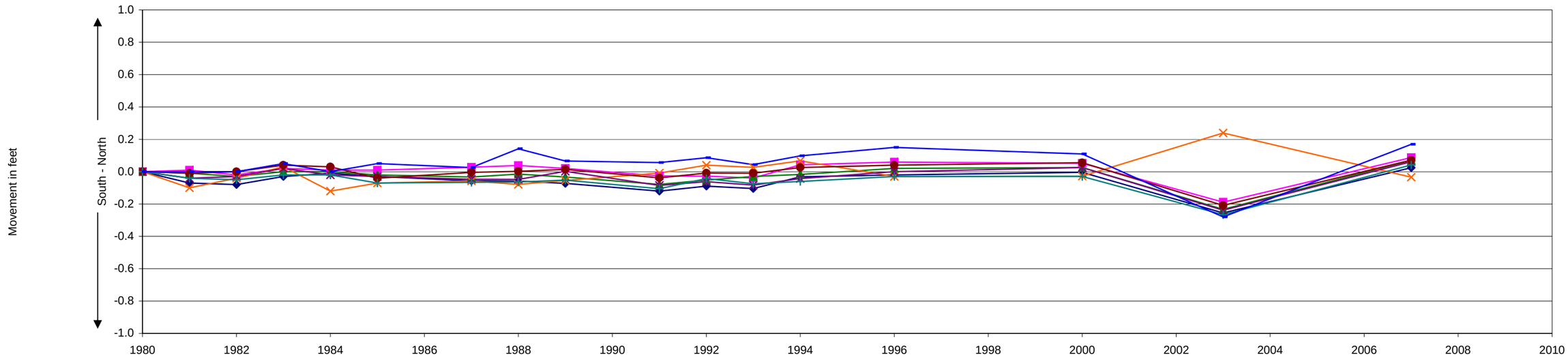
Re-Allocation Study Plate D-5

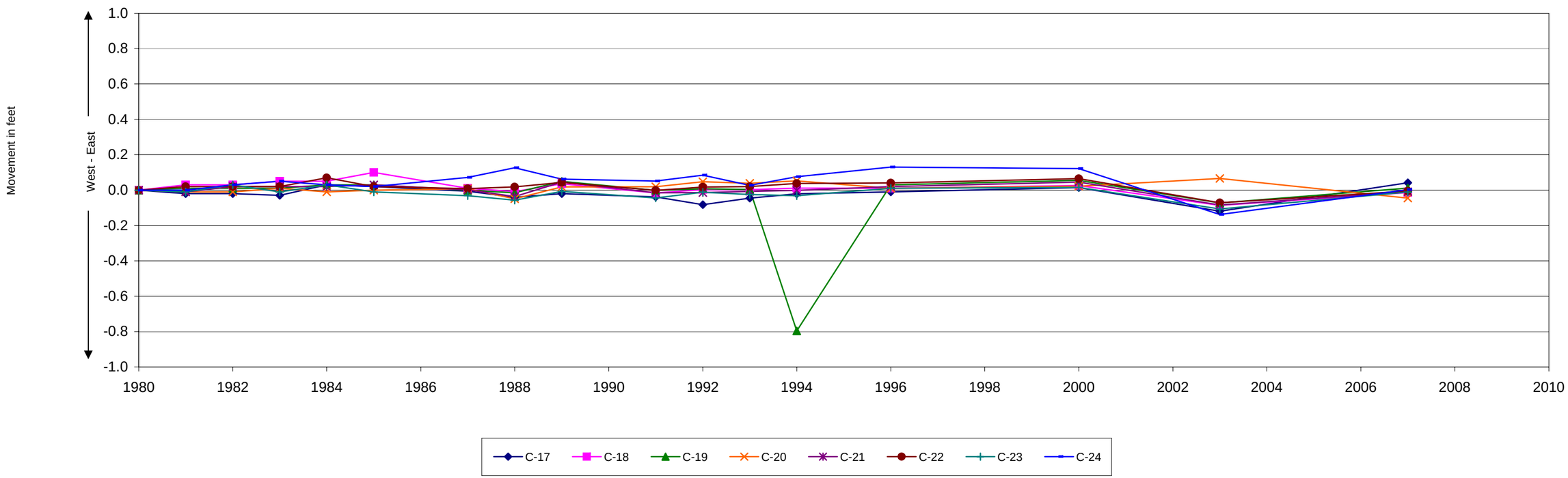




Chatfield Dam and Lake, CO
Embankment & Foundation Movement
Crest, Slope, and Toe Movement Markers
Horizontal Movement vs. Time Plot Crest Markers 9-16

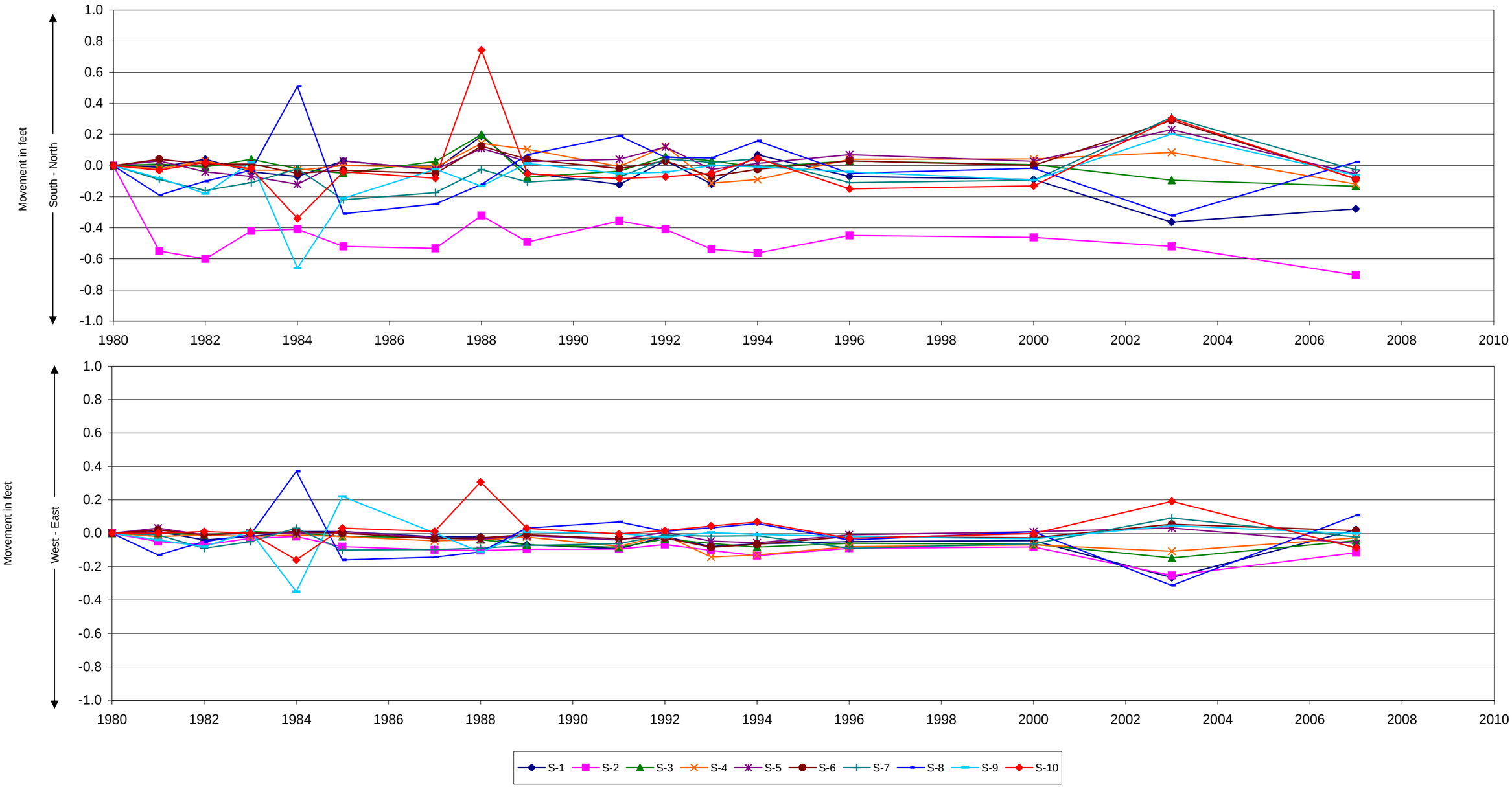
Re-Allocation Study Plate D-6





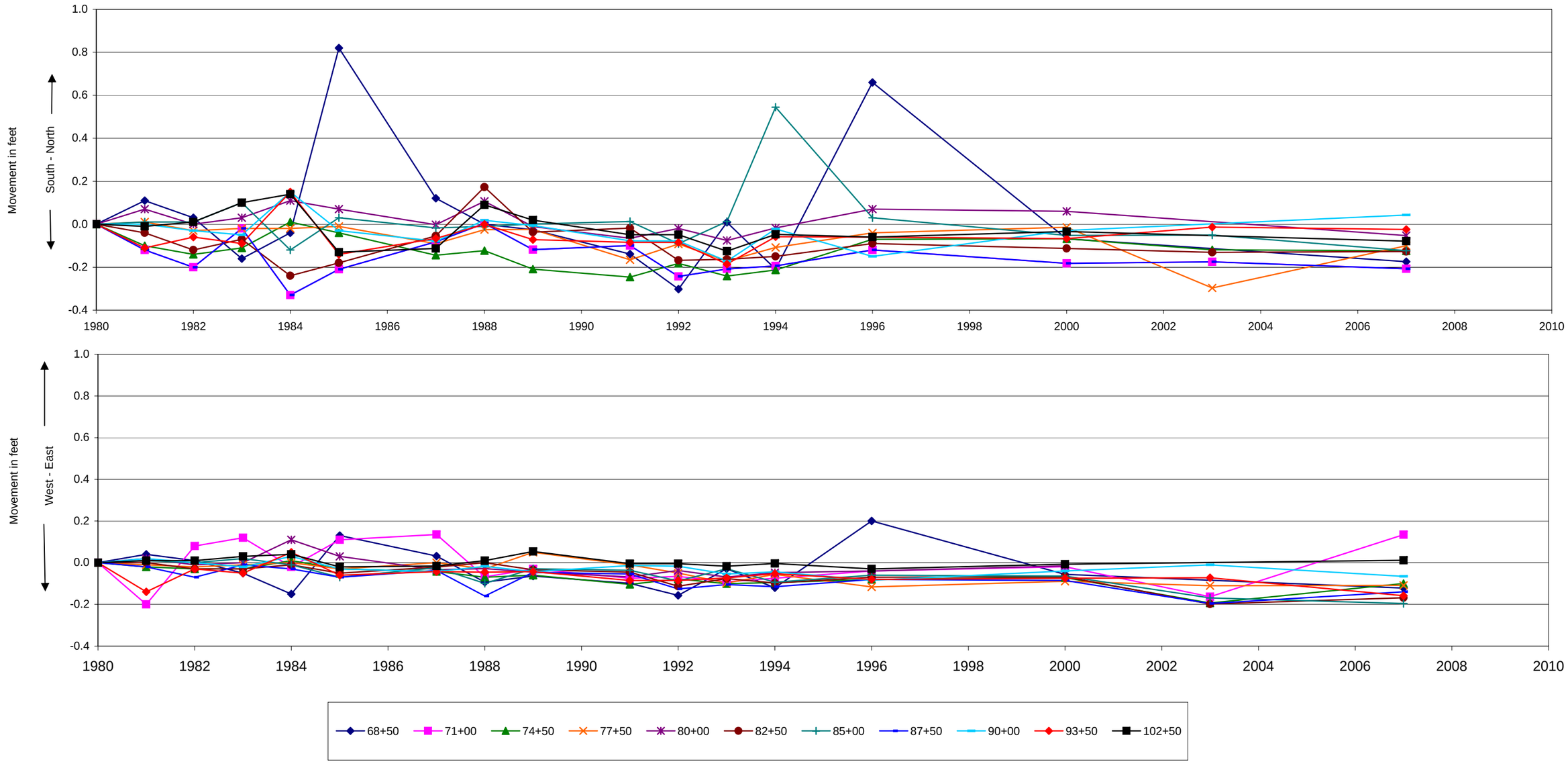
Chatfield Dam and Lake, CO
Embankment & Foundation Movement
Crest, Slope, and Toe Movement Markers
Horizontal Movement vs Time Plot Crest Makers 17-24

Re-Allocation Study Plate D-7



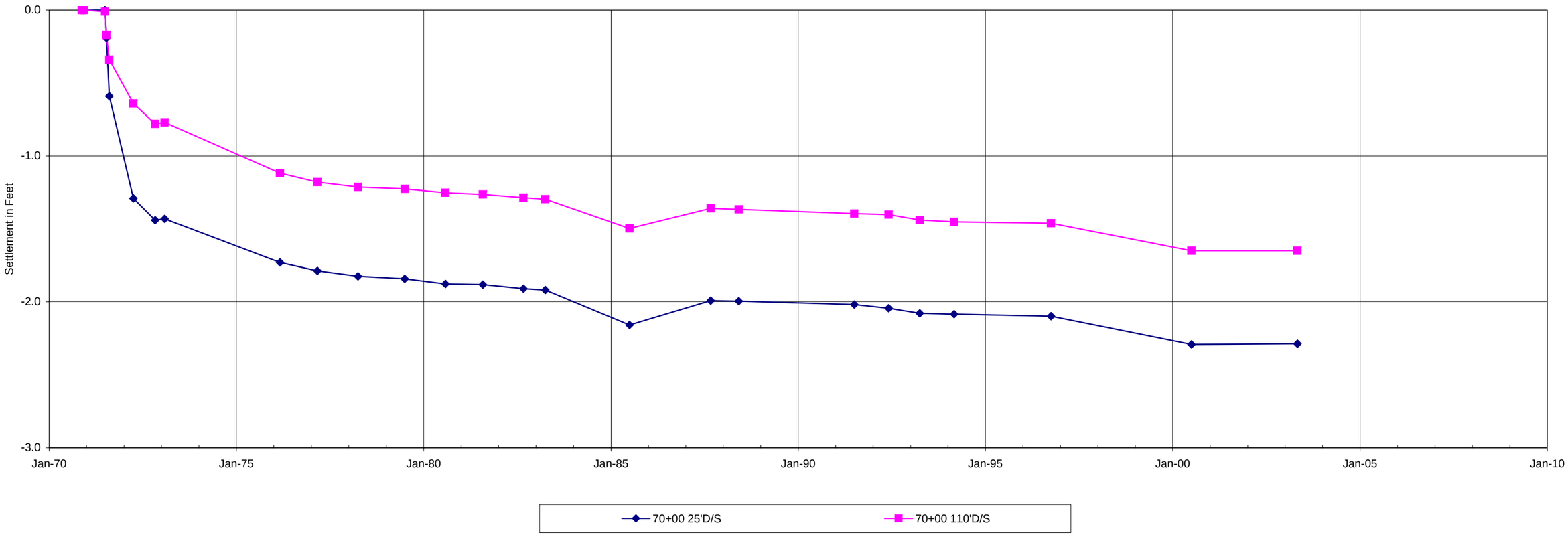
Chatfield Dam and Lake, CO
Embankment & Foundation Movement
Crest, Slope, and Toe Movement Markers
Horizontal Movement vs. Time Plot

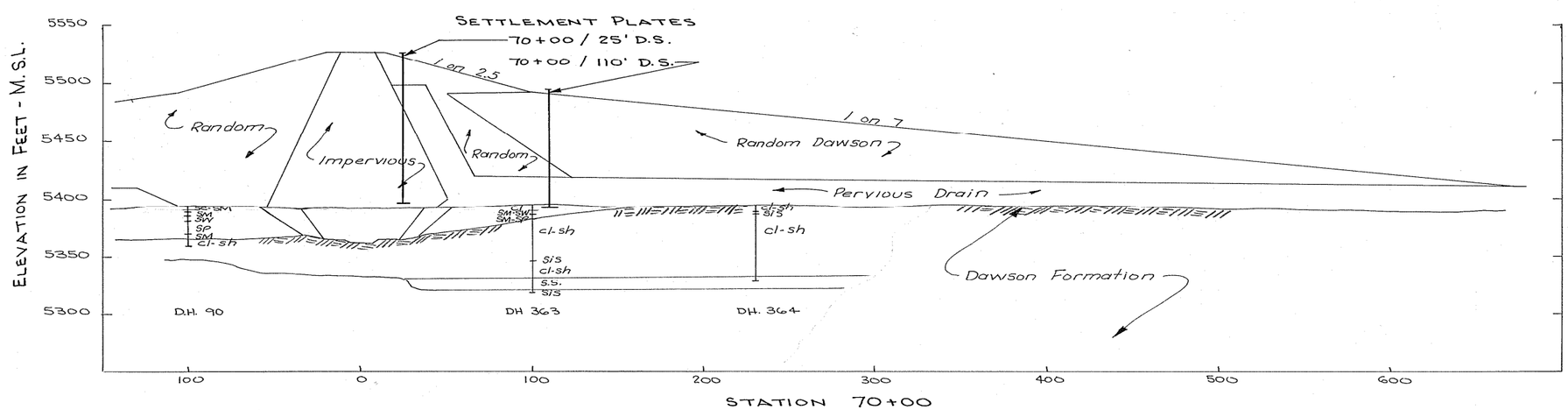
Re-Allocation Study Plate D-8



Chatfield Dam and Lake, CO
Embankment & Foundation Movement
Crest, Slope, and Toe Movement Markers
Horizontal Movement vs. Time Plot Toe Markers

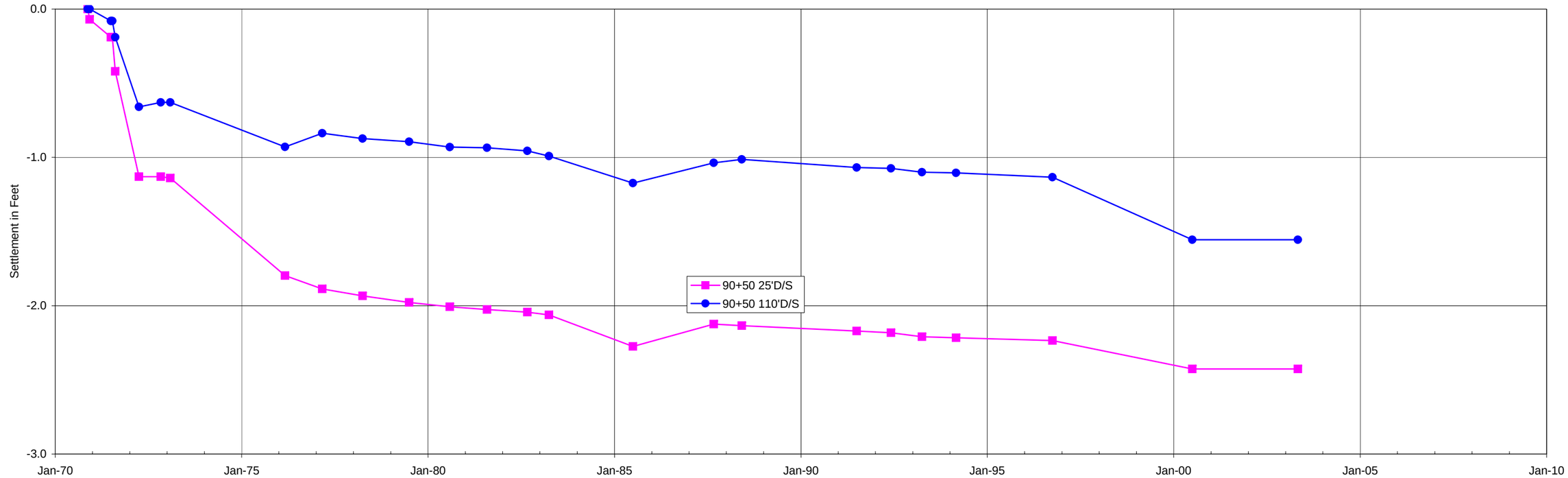
Re-Allocation Study Plate D-9

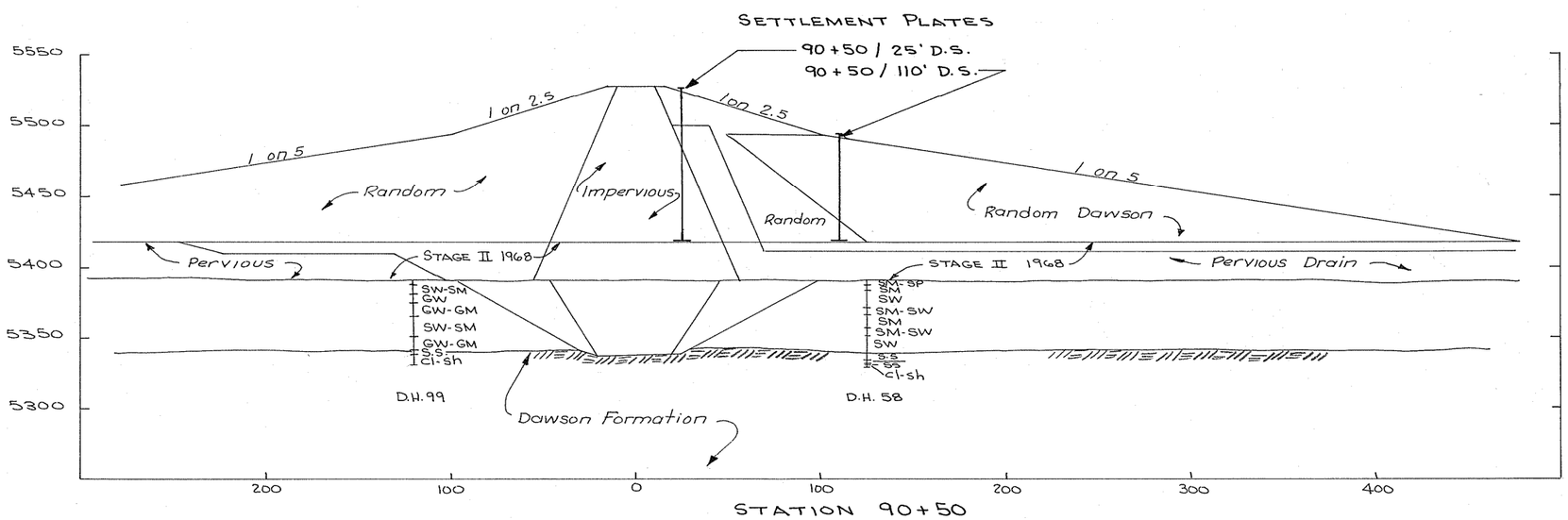




Chatfield Dam and Lake, CO
Embankment & Foundation Movement
Settlement Gages - Sta. 70+00
Vertical Movement

Re-Allocation Study Plate D-10





Chatfield Dam and Lake, CO
Embankment & Foundation Movement
Settlement Gages - Sta. 90+00
Vertical Movement

Re-Allocation Study Plate D-11

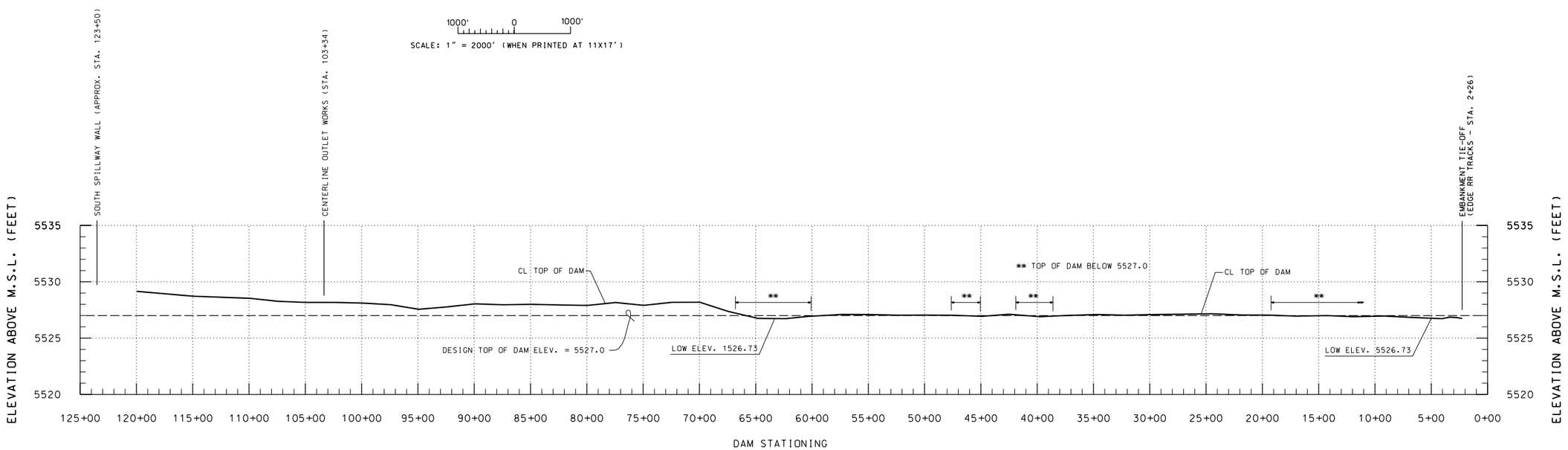
NOTES:

1. STATIONING SHOWN IS APPROXIMATELY EQUIVALENT TO DAM STATIONING. TOP OF DAM SURVEY WAS VISUALLY LINED UP WITH SEPT 2007 AERIAL PHOTOGRAPH OF DAM. STATIONING WAS ESTIMATED BASED ON CENTERLINE OF OUTLET WORKS.
2. TOP OF DAM SURVEY WAS CONDUCTED MAY 2007.
3. VERTICAL DATUM IS 1929 NGVD.
4. VERTICAL CONTROL FOR SURVEY WAS CONTROL POINT "WEST".

PLAN VIEW - CHATFIELD DAM

1000' 0 1000'

SCALE: 1" = 2000' (WHEN PRINTED AT 11X17')



TOP OF DAM PROFILE

500' 0 500'

5' 0 5'

HOR. SCALE: 1" = 1000' (WHEN PRINTED AT 11X17')
VER. SCALE: 1" = 10' (WHEN PRINTED AT 11X17')

US ARMY CORPS OF ENGINEERS
OMAHA DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

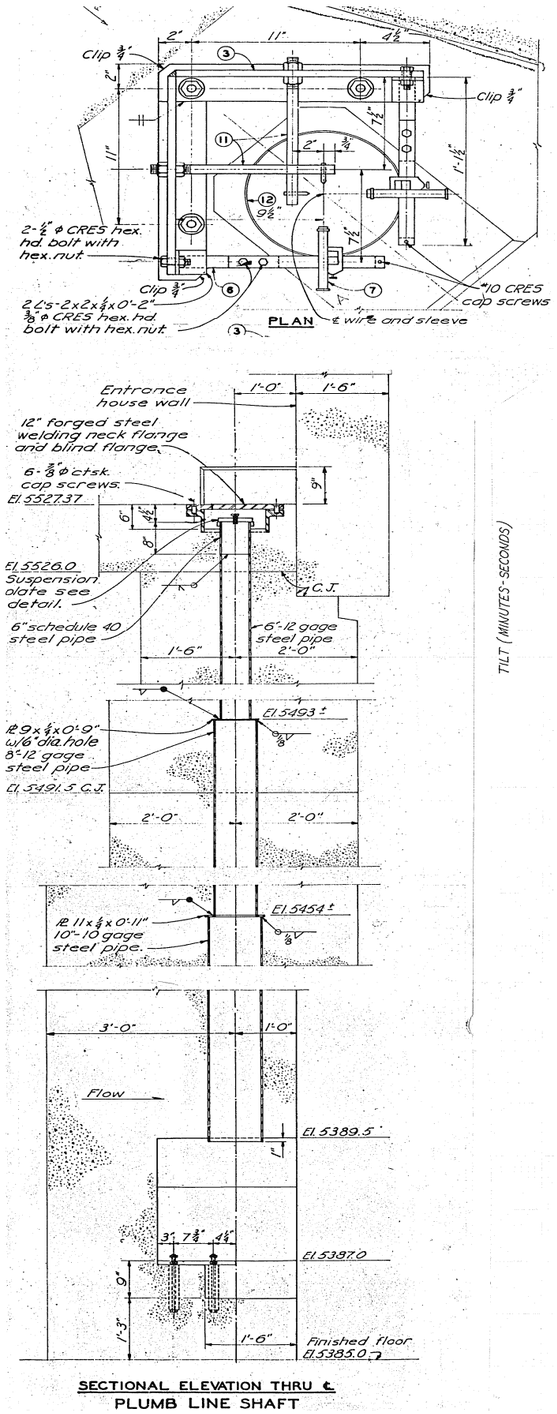
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
OMAHA DISTRICT

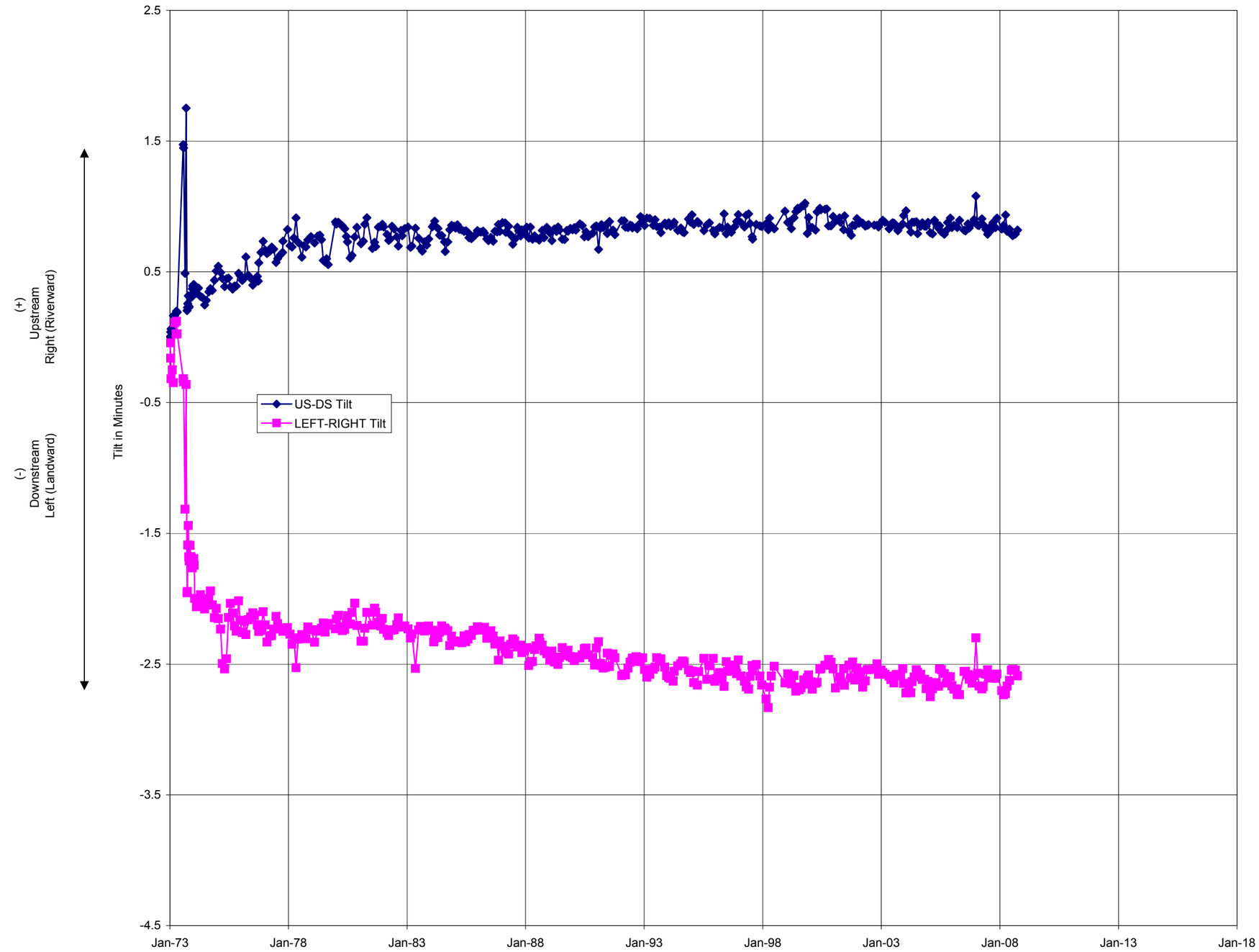
DESIGNED BY: | DATE: 20 OCT 2008
DWN BY: B.J.L. | CKD BY: M.T.K. | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
FILE NAME: | FILE NUMBER:
SIZE: | PLOT SCALE: | PLOT DATE:

CHATFIELD DAM AND LAKE, COLORADO

TOP OF DAM SURVEY
MAY 2007

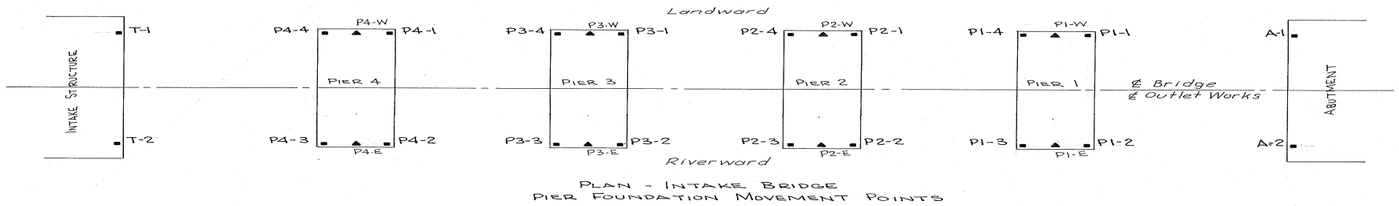
SHEET IDENTIFICATION NUMBER
D-12



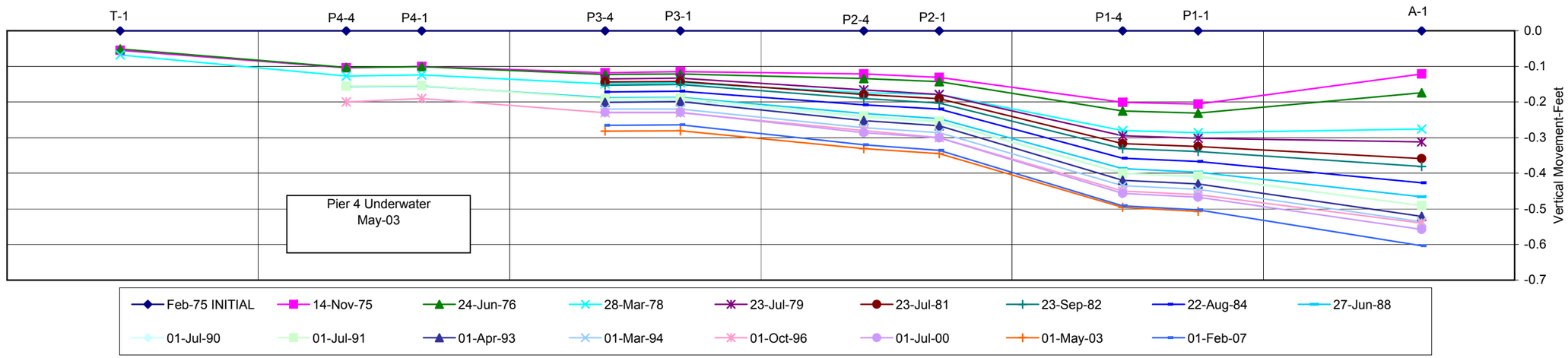


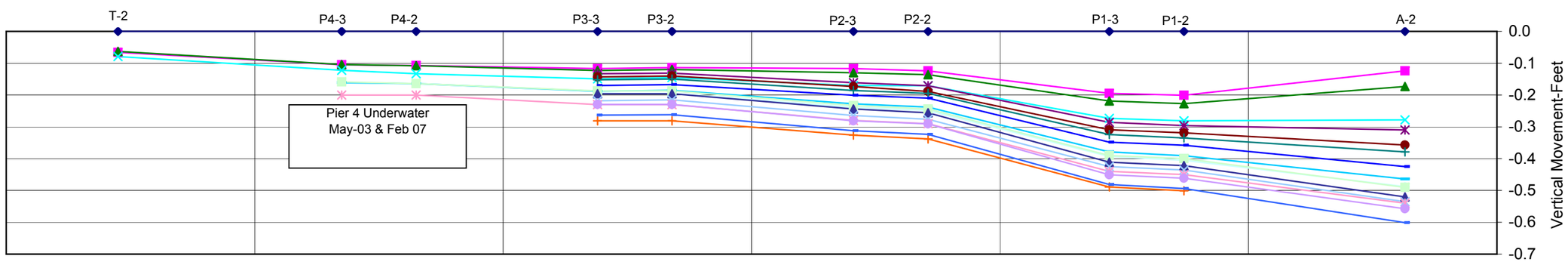
Chatfield Dam and Lake, Colorado
Outlet Works - Intake Structure
Plumbline Tilt

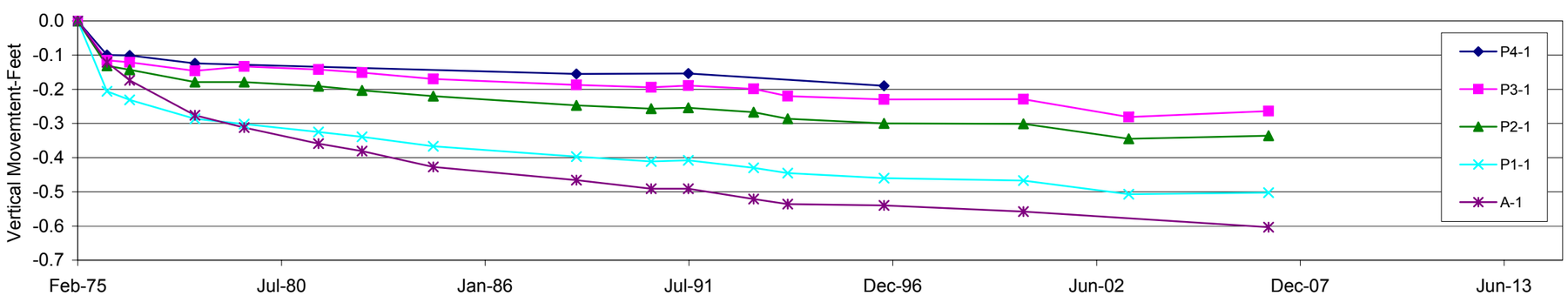
Re-Allocation Study Plate D-13



Plan - Intake Bridge
Pier Foundation Movement Points

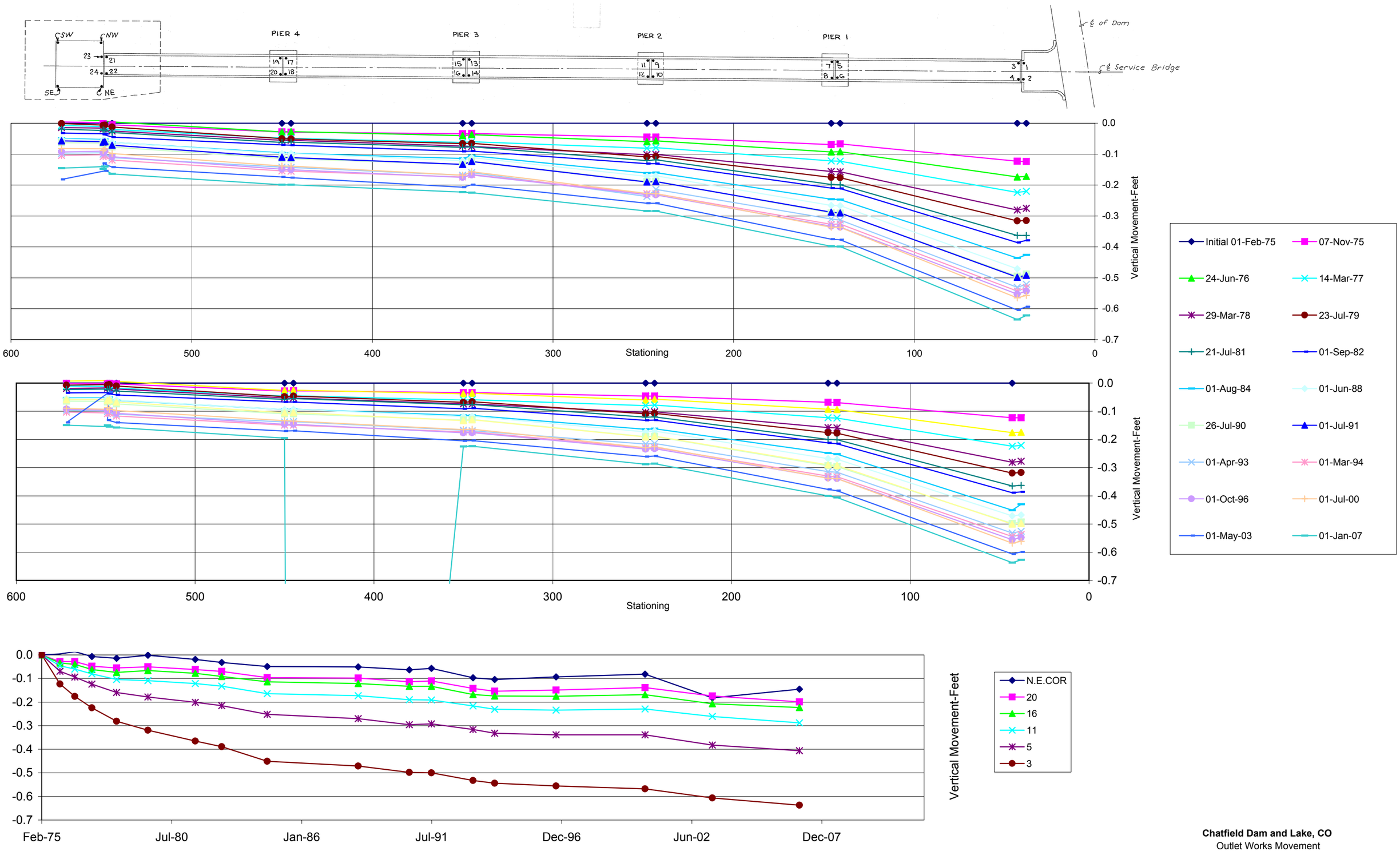






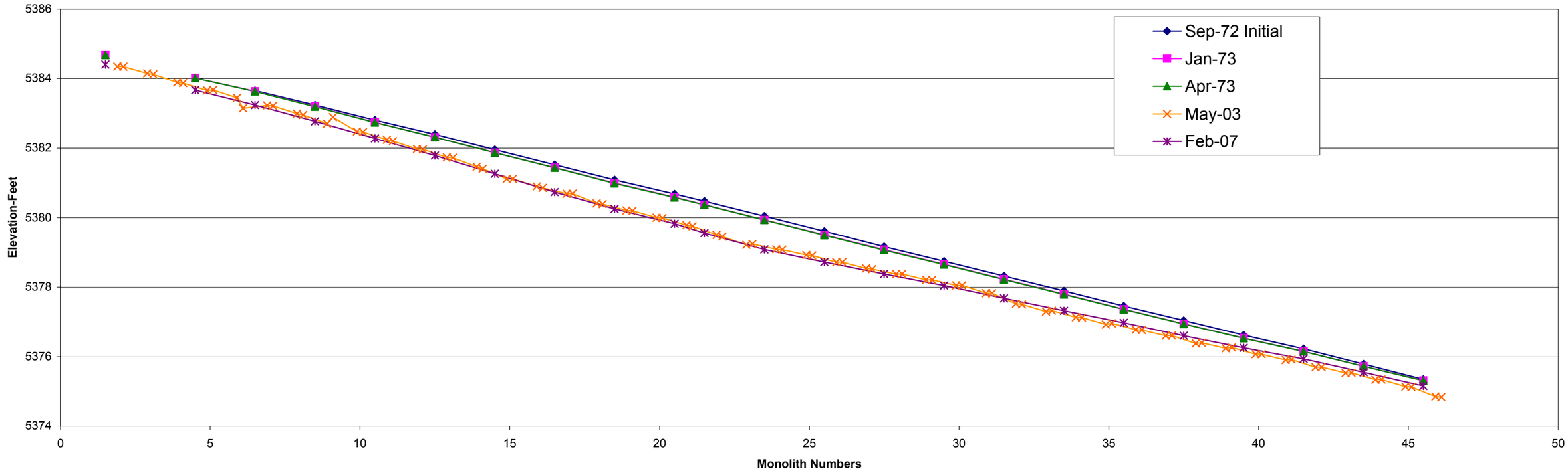
Chatfield Dam and Lake, CO
Outlet Works Movement
Intake Bridge Piers & Tower
Verical Movement

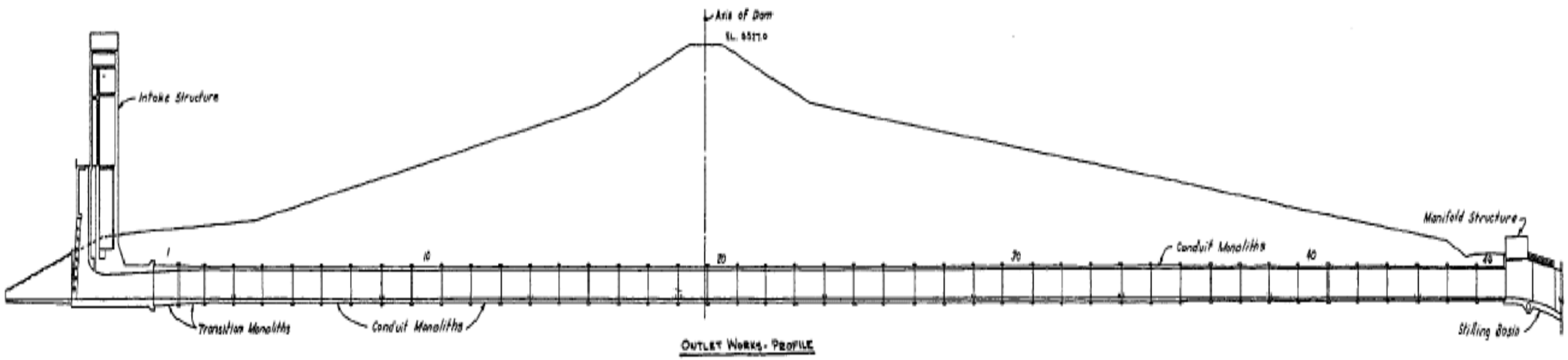
Re-Allocation Study Plate D-14



Chatfield Dam and Lake, CO
Outlet Works Movement
Intake Bridge Deck & Tower Vertical Movement

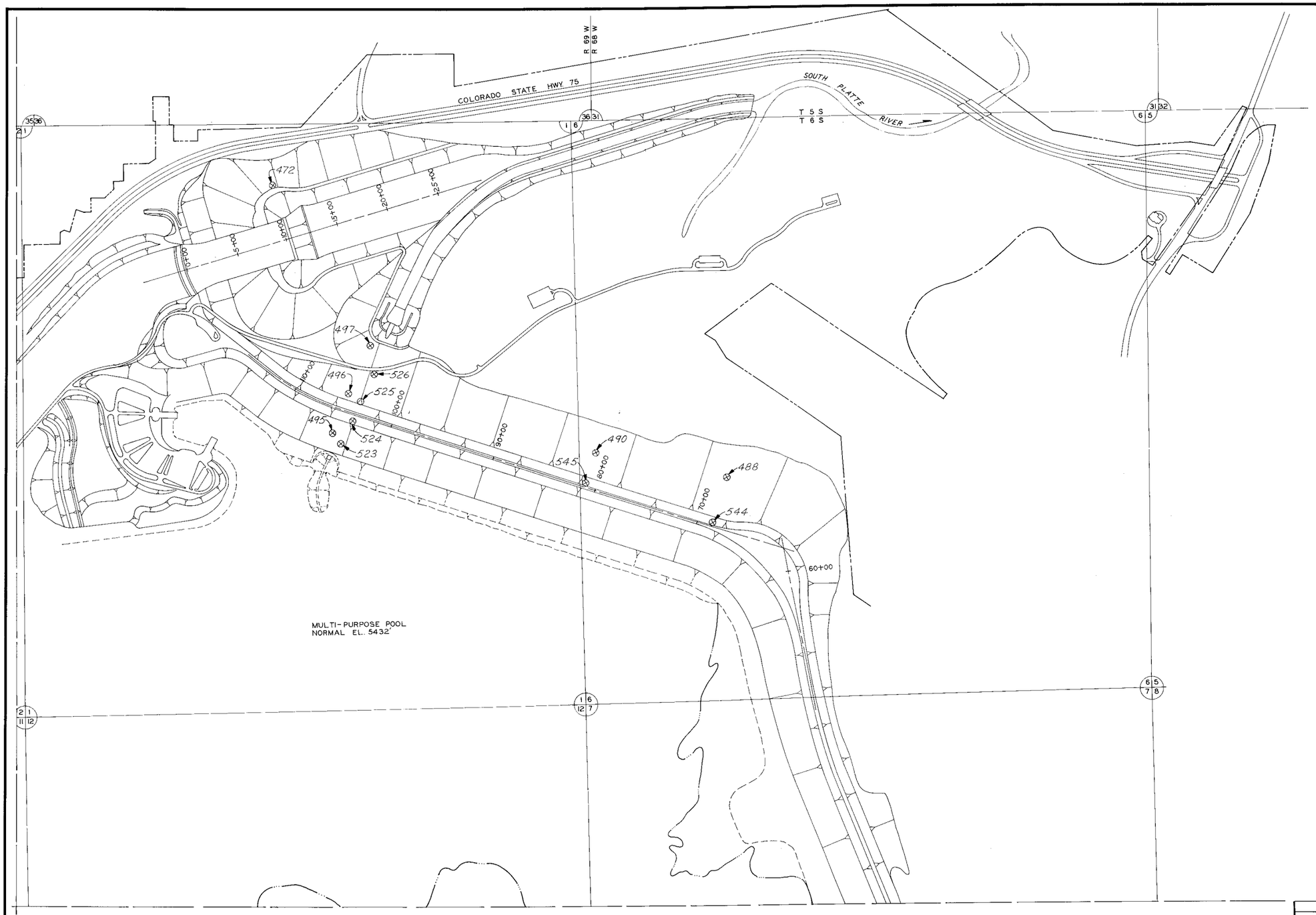
Re-Allocation Study Plate D-15





Chatfield Dam
Outlet Works - Wet Conduit
Vertical Movement Profile

Re-Allocation Study Plate D-16



INCLINOMETERS		
EMBANKMENT		
NO.	STATION	RANGE
544	68 + 80	17' D. S.
488	68 + 80	500' D. S.
545	81 + 20	17' D. S.
490	81 + 20	350' D. S.
495	104 + 78.1	232' U. S.
496	104 + 53.9	167.2' D. S.
497	104 + 21.2	667.5 D. S.
OUTLET WORKS		
523	-2 + 92 (U. S.)	9.5' Left
524	-0 + 68 (U. S.)	9.5' Left
525	1 + 00 (D. S.)	9.5' Left
526	4 + 36 (D. S.)	9.5' Left
SPILLWAY SLOPE		
472	10 + 60	500' Left

SCALE: 1 INCH = 400 FEET
400' 0 400'

LEGEND:
⊕ Inclinometers

DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

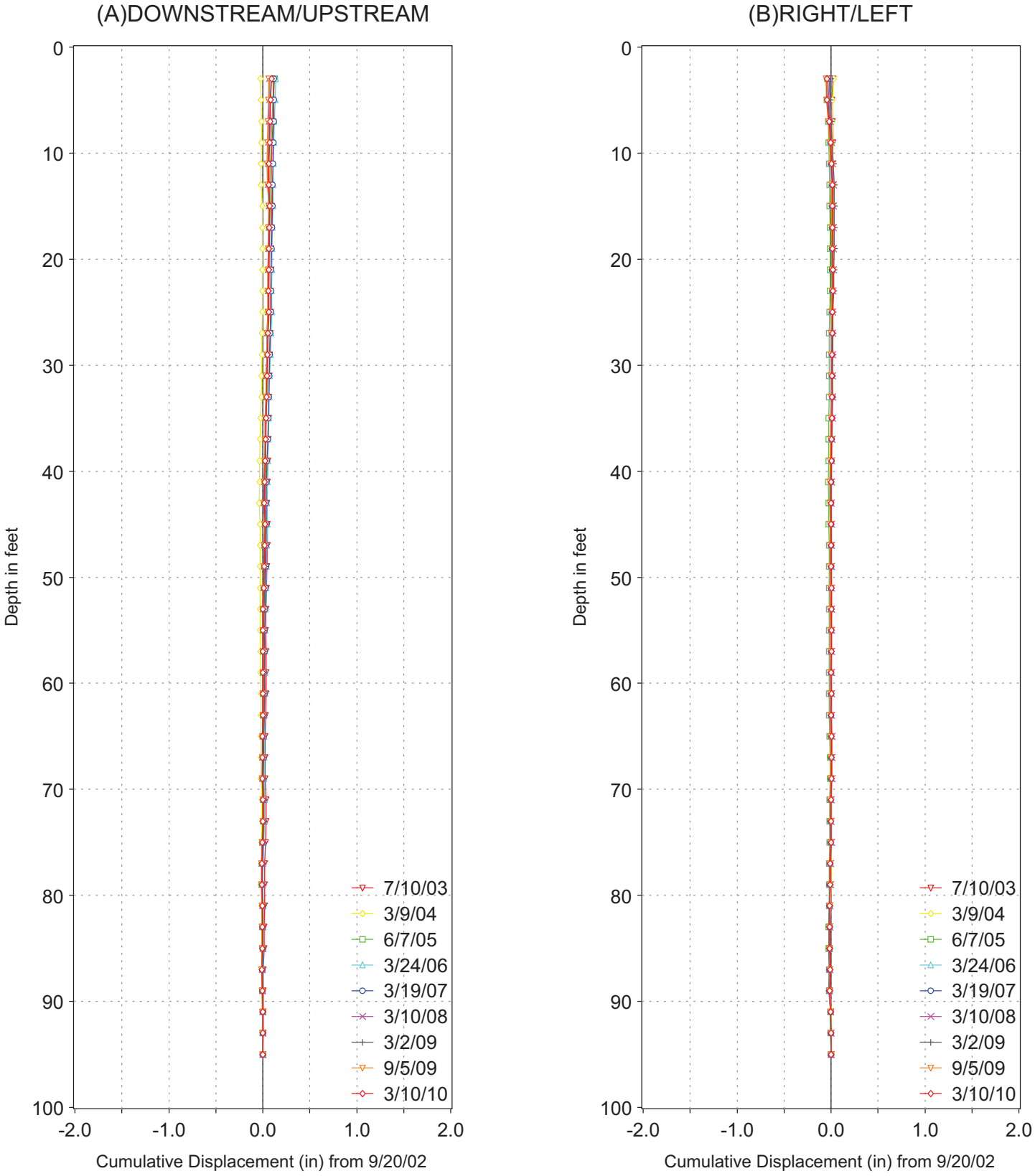
DESIGNED BY:
DRAWN BY: J. L. C.
CHECKED BY:
SUBMITTED BY:
CHIEF SECTION
RECOMMENDED:
CHIEF DESIGN BRANCH
APPROVED:
COL. C. E. DISTRICT ENGINEER

SOUTH PLATTE RIVER
CHATFIELD LAKE, COLORADO
EMBANKMENT AND SPILLWAY
INCLINOMETERS
LOCATION PLAN

APPROVED: CHIEF ENGINEERING DIVISION
DATE:
SCALE: AS SHOWN
SPEC. NO. DACA45
DRAWING NUMBER
SHEET 1

THIS PLAN ACCOMPANIES CONTRACT No. DACA45 MODIFICATION No.

U. S. GOVERNMENT PRINTING OFFICE: 1976-646-144



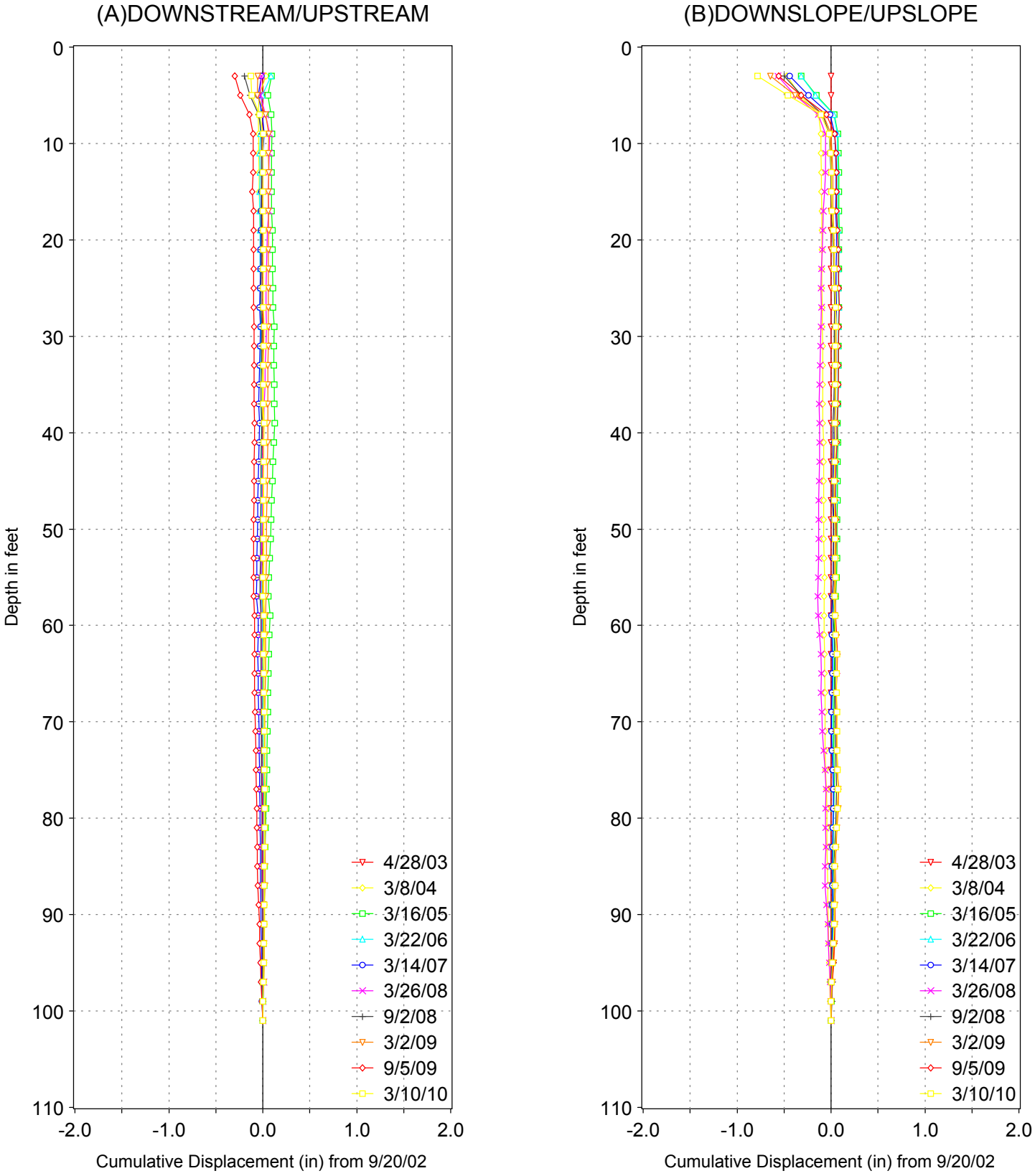
US Army Corps of Engineers®

Chatfield Dam and Lake, CO

Inclinometer 497, Sta. 104+21, 668' DS

Cumulative Displacement 2002 - Present

Reallocation Study **Plate D-18**



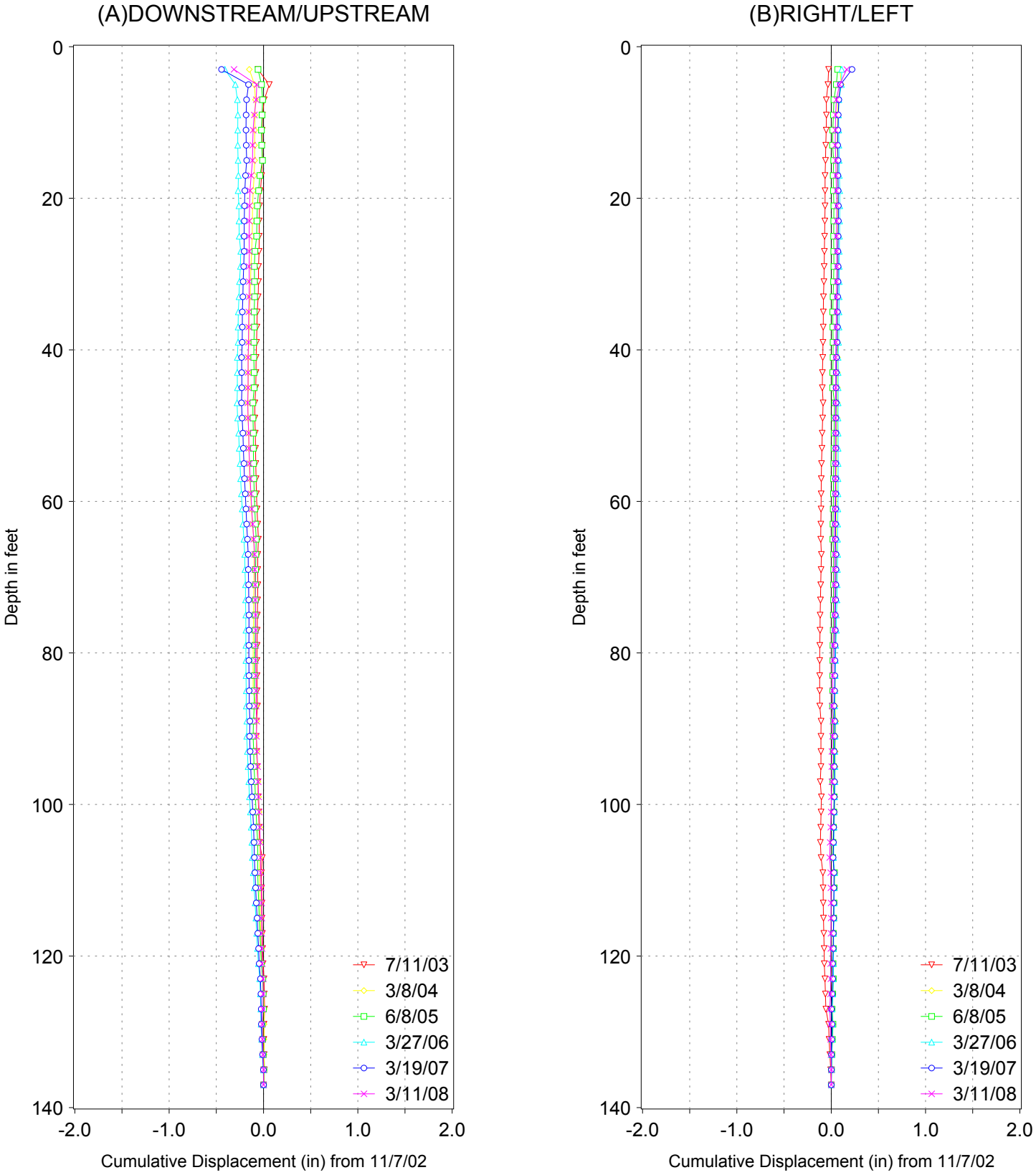
US Army Corps of Engineers®

Chatfield Dam and Lake, CO

Inclinometer 472, Spillway Sta. 10+60, 500' Left

Cumulative Displacement 2002-Present

Reallocation Study **Plate D-19**



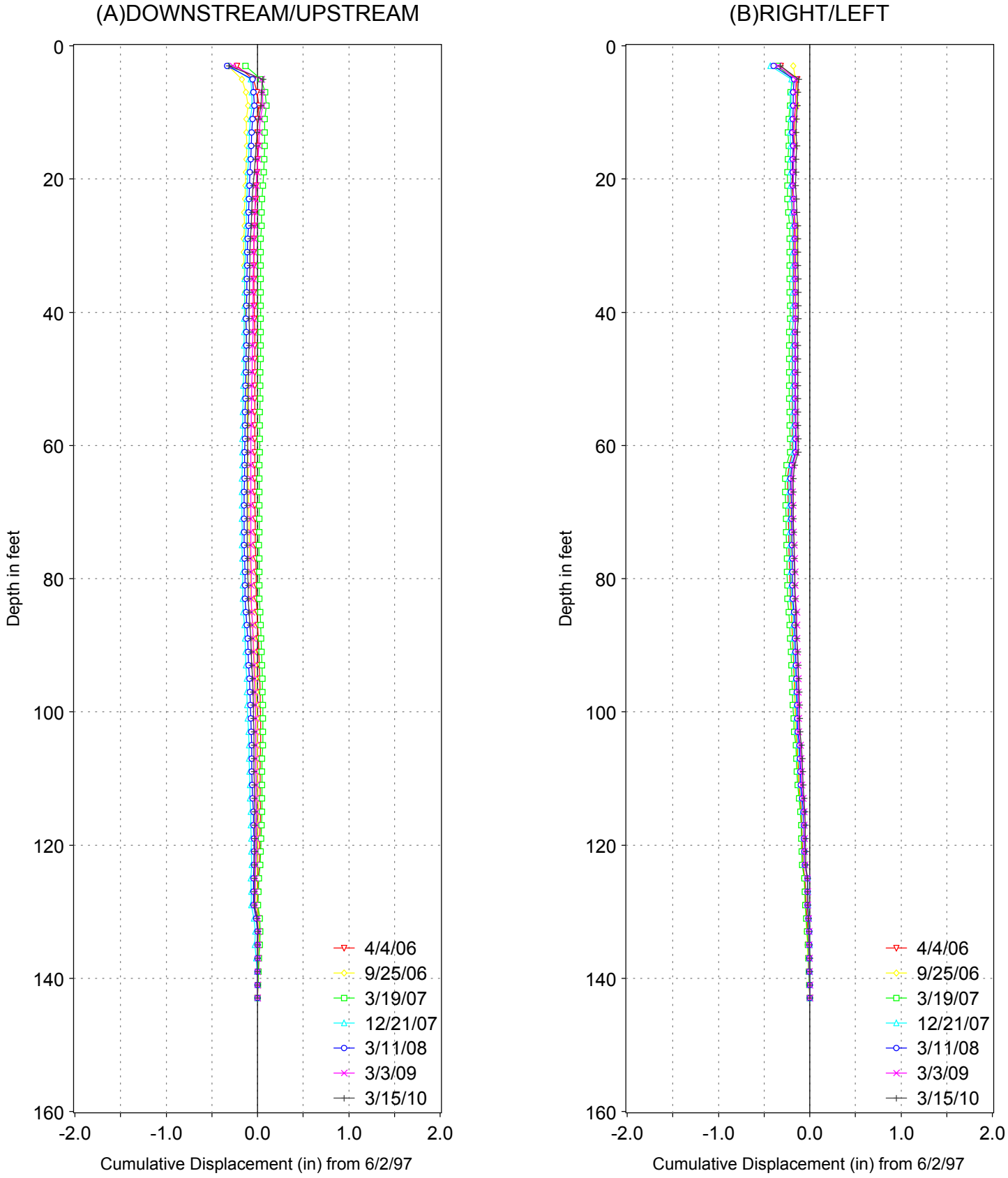
US Army Corps of Engineers®

Chatfield Dam and Lake, CO

Inclinometer 488, Sta. 68+90, 500' DS

Cumulative Displacement 2002 - Present

Realloction Study **Plate D-20**



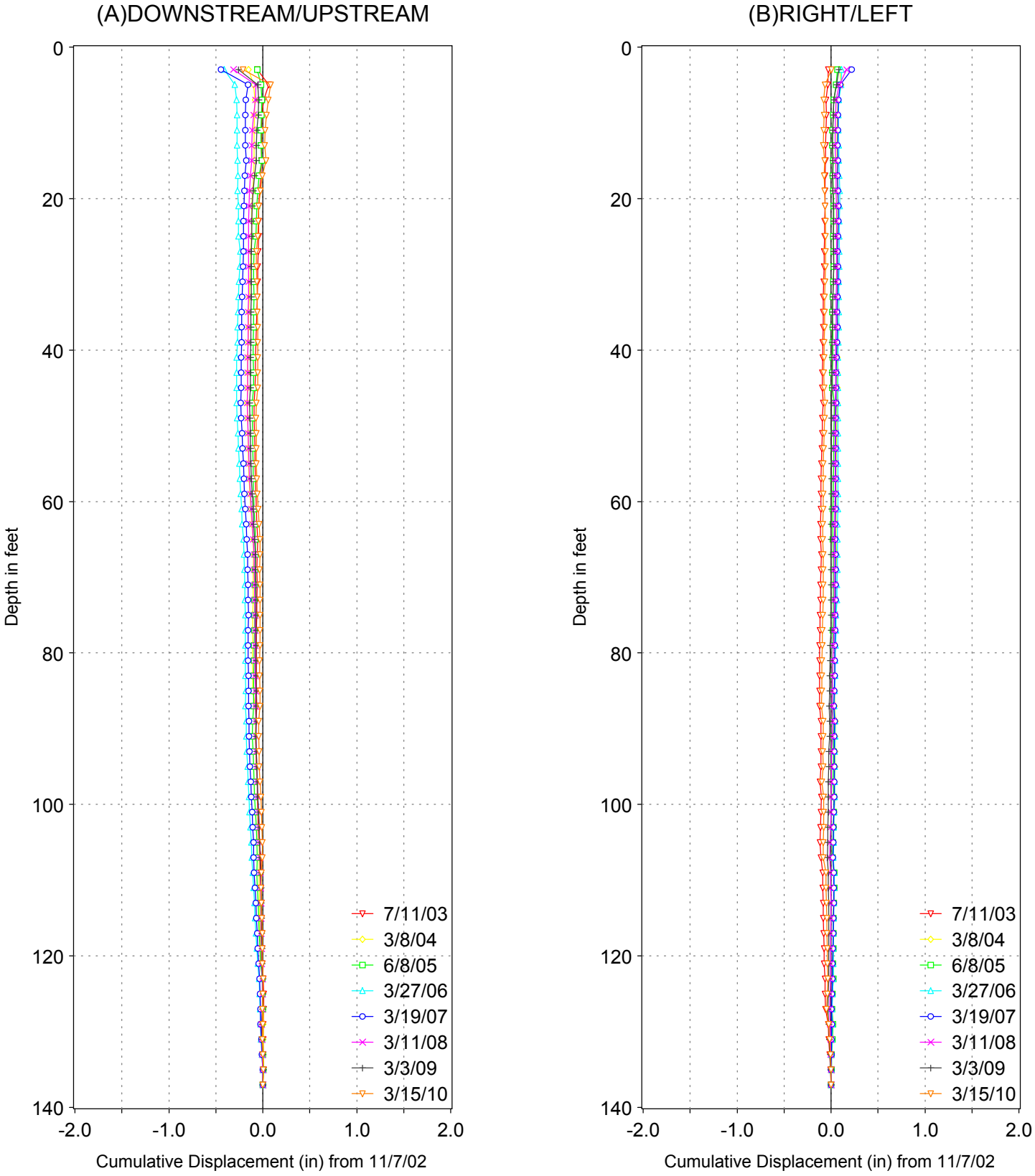
US Army Corps of Engineers®

Chatfield Dam and Lake, CO

Inclinometer 490, Sta. 81+20, 350' DS

Cumulative Displacement 97 - Present

Reallocation Study **Plate D-21**



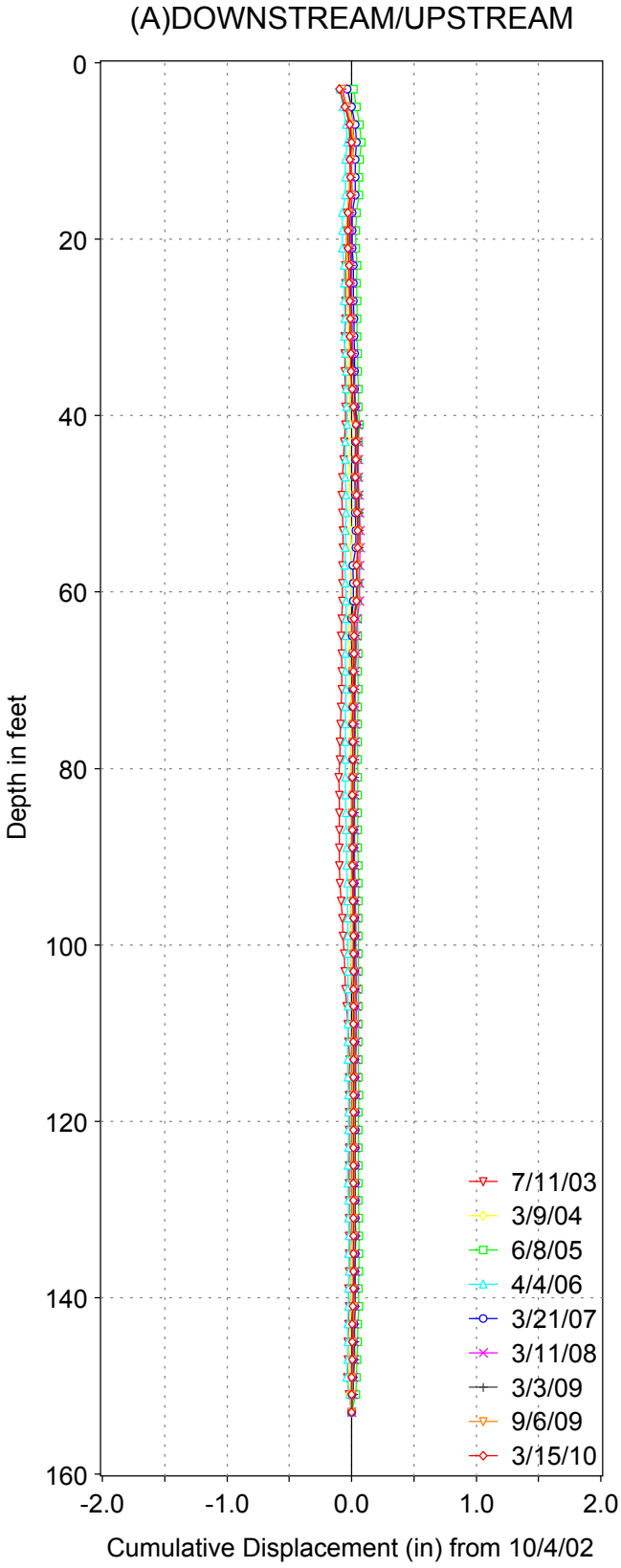
US Army Corps of Engineers®

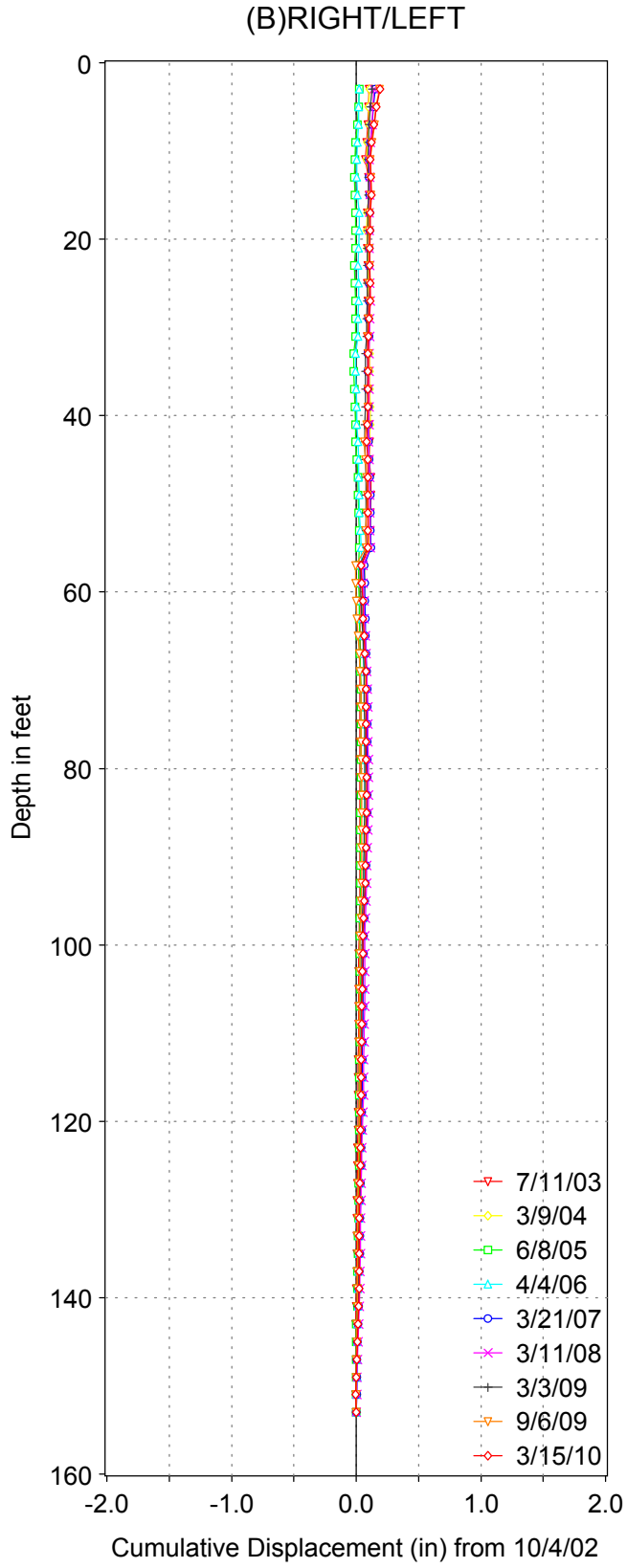
Chatfield Dam and Lake, CO

Inclinometer 488, Sta. 68+90, 500' DS

Cumulative Displacement 2002 - Present

Reallocation Study **Plate D-22**





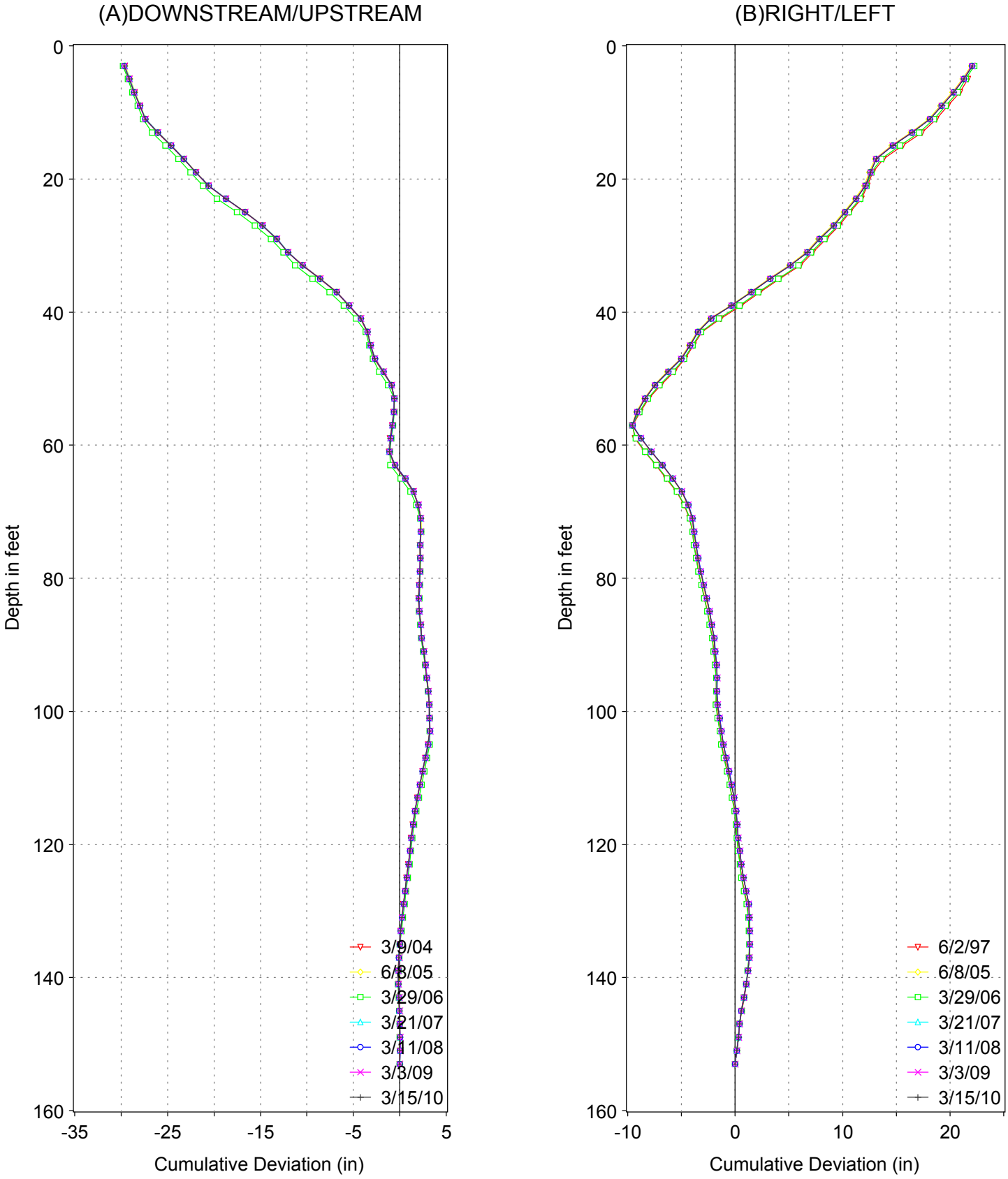
US Army Corps of Engineers®

Chatfield Dam and Dam, CO

Inclinometer 496, Sta. 104+54, 167' DS

Cumulative Displacement 2002 - Present

Reallocation Study **Plate D-23**



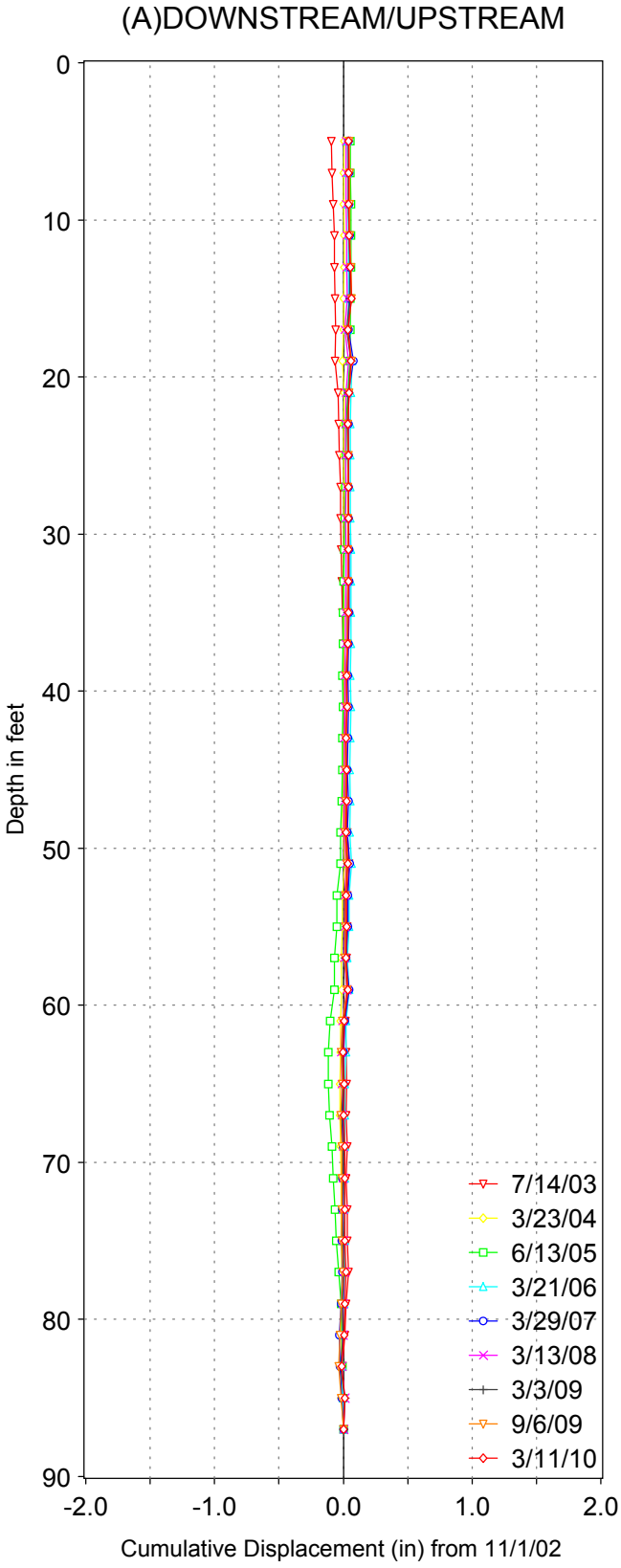
US Army Corps of Engineers®

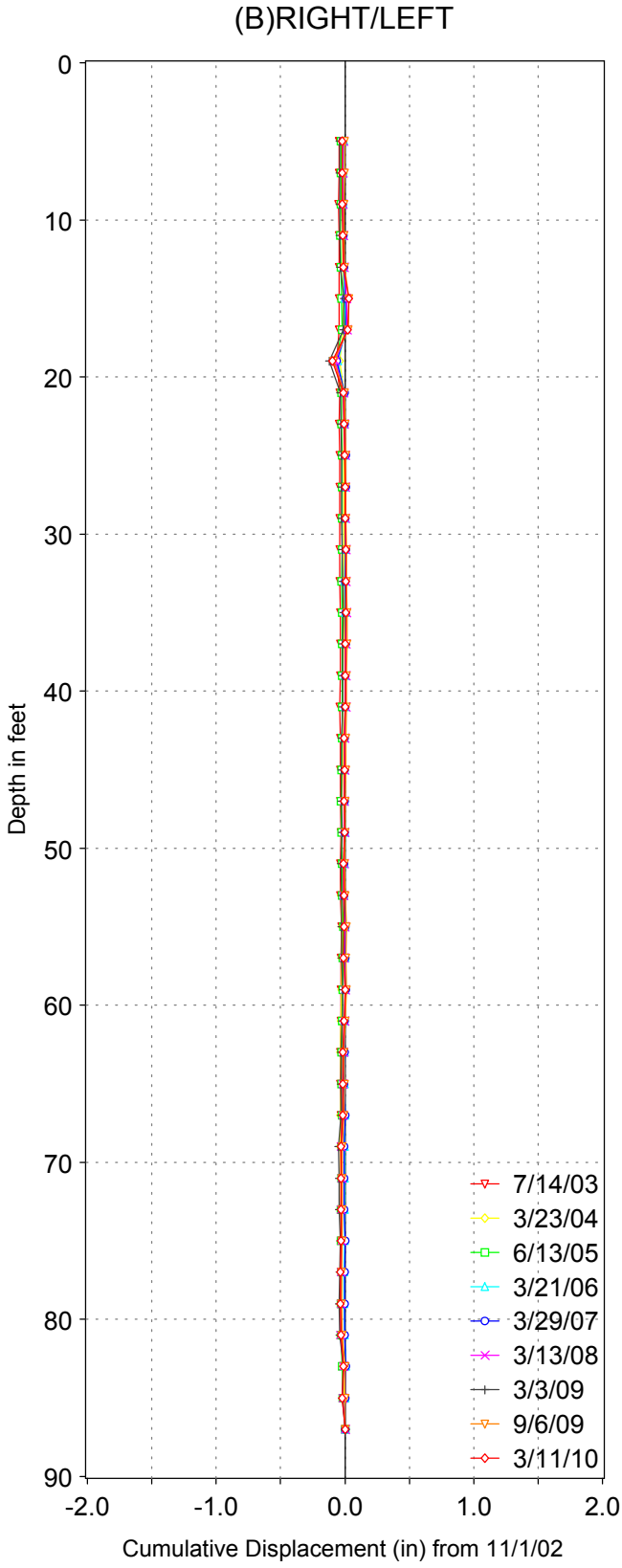
Chatfield Dam and Lake, CO

Inclinometer 496, Sta. 104+54, 167' DS

Cumulative Deviation

Reallocation Study **Plate D-23A**





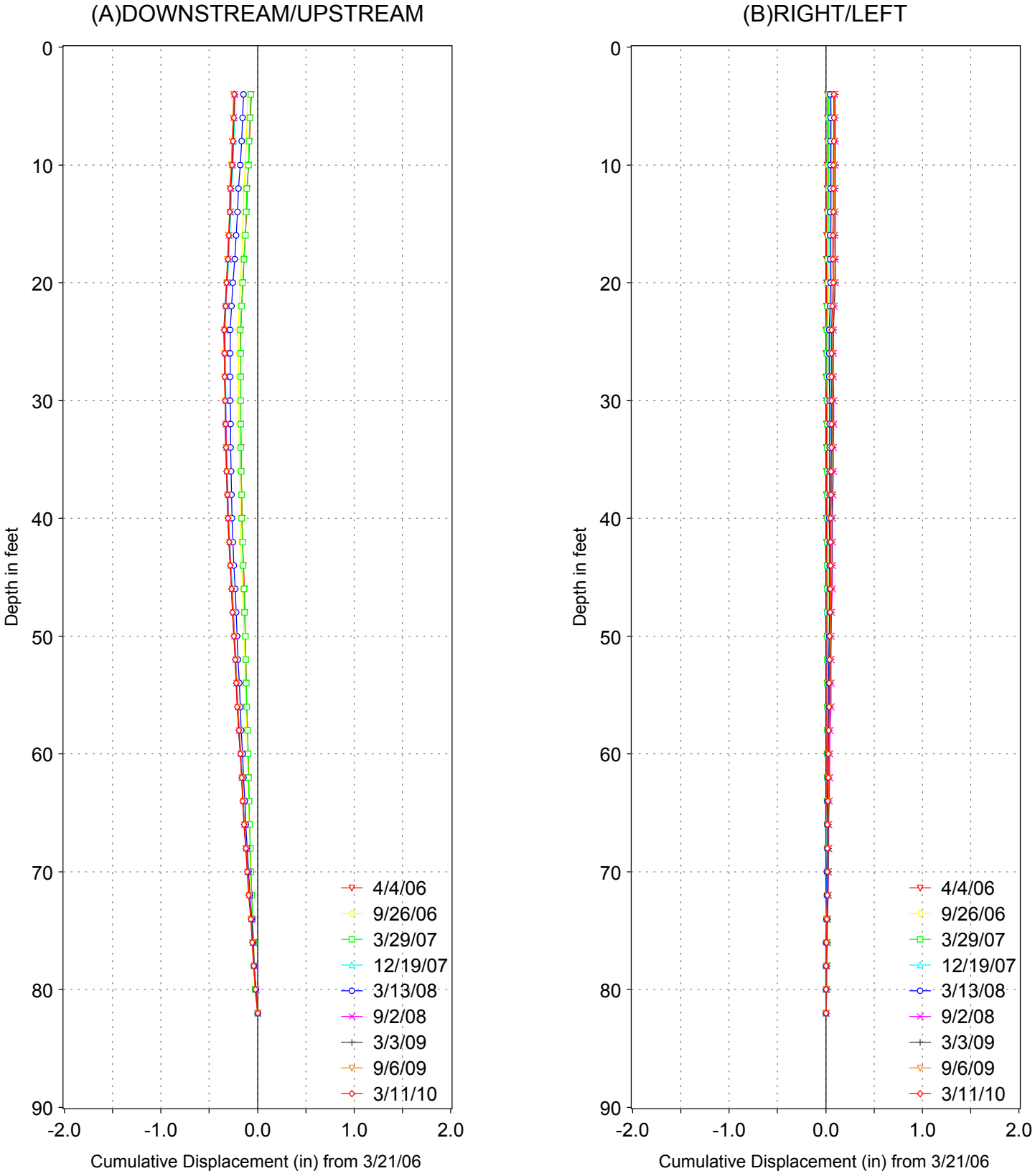
US Army Corps of Engineers®

Chatfield Dam and Lake, CO

Inclinometers 523, Outlet Works Sta. 2+92, 9.5' LEFT

Cumulative Displacement 2002 - Present

Realloction Study **Plate D-24**



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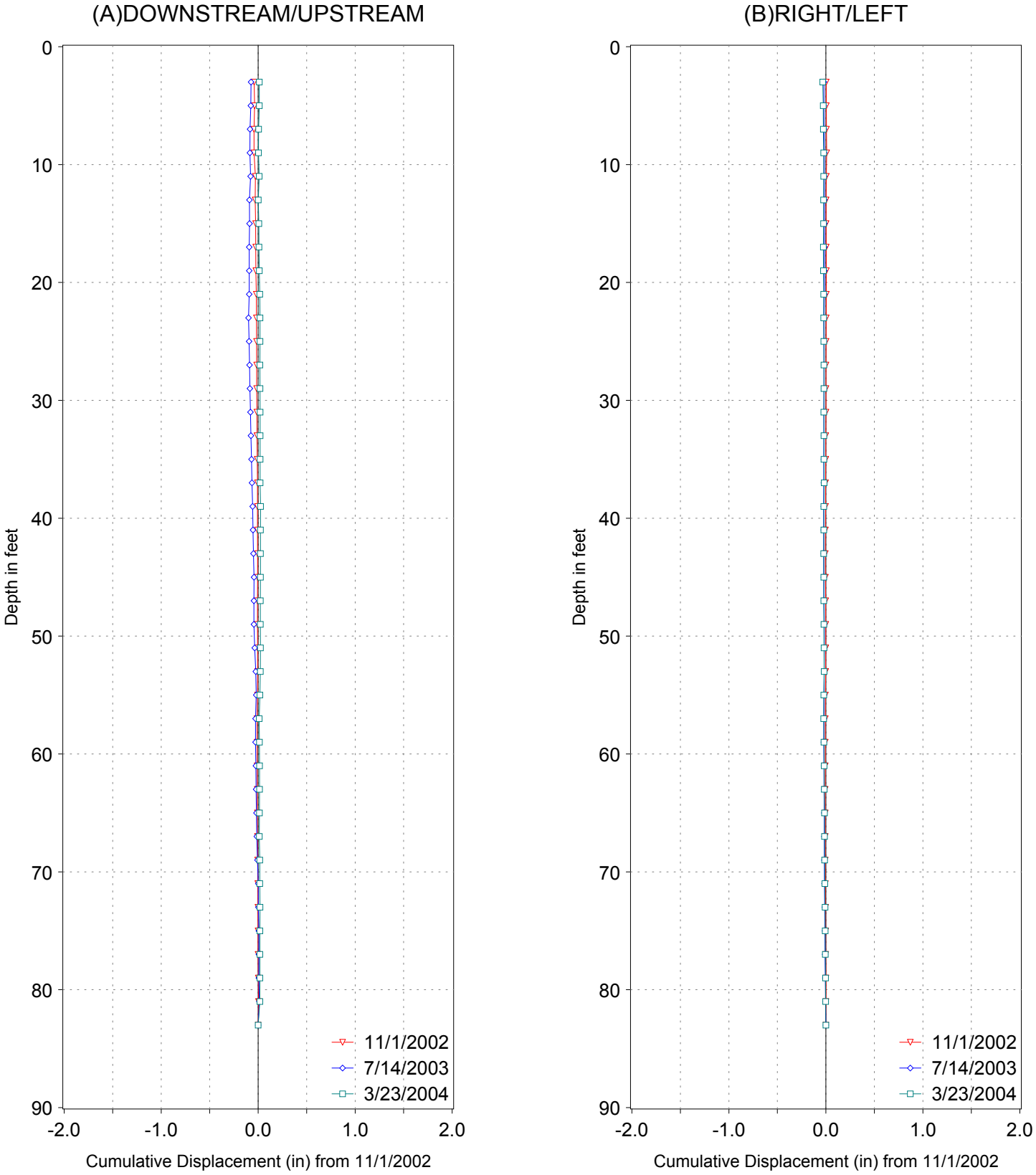
Chatfield Dam and Lake, CO

Inclinometer 524, Outlet Works Sta. 0+68, 9.5' Left

Cumulative Displacement 2006 - Present

Reallocation Study **Plate B-25**



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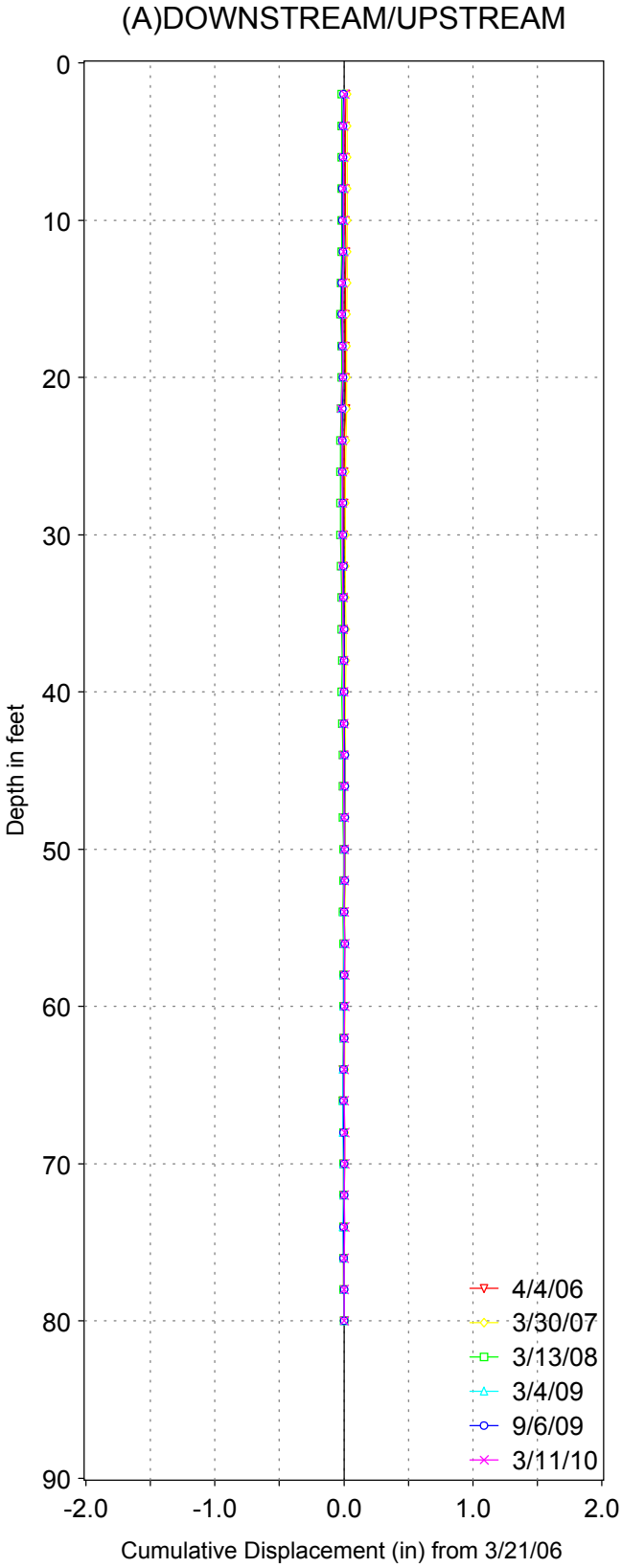
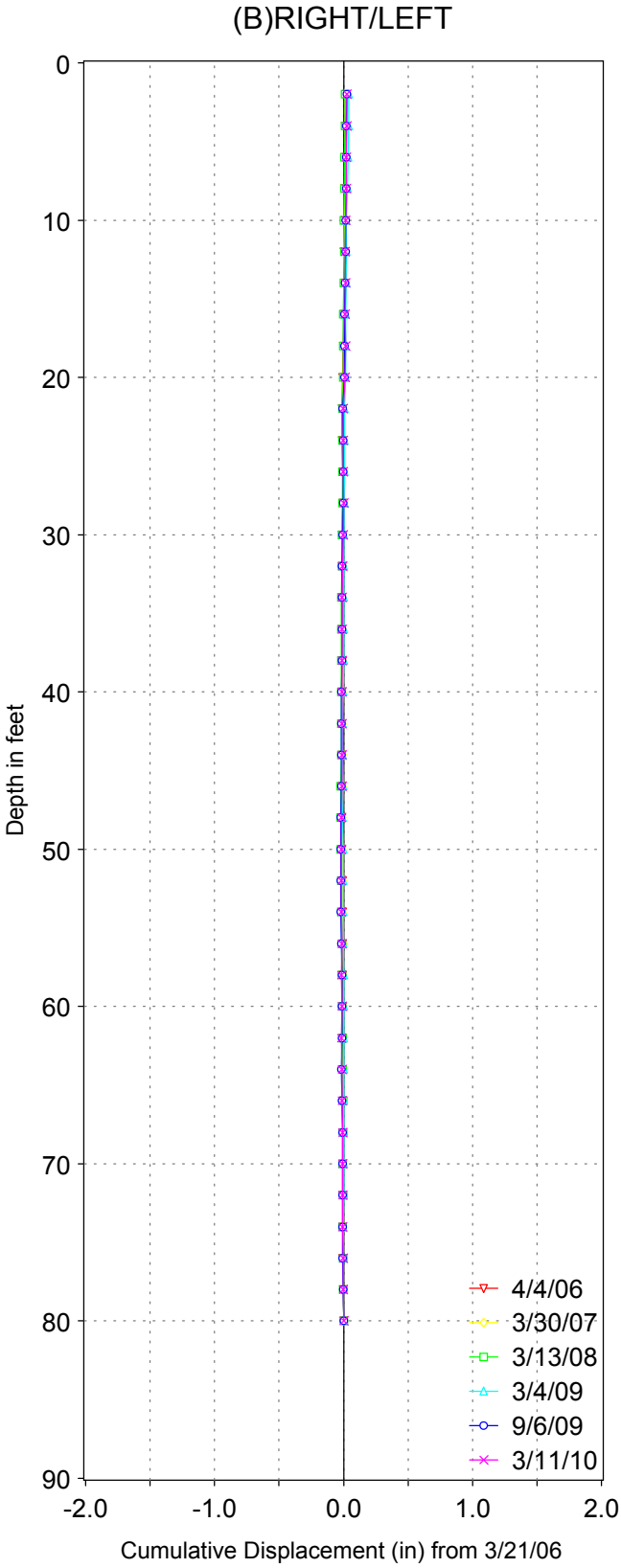
Inclinometer 525, Outlet Works Sta. 1+00, 9.5' Left

Cumulative Displacement 2002 - Present

Reallocation Study **Plate D-26**





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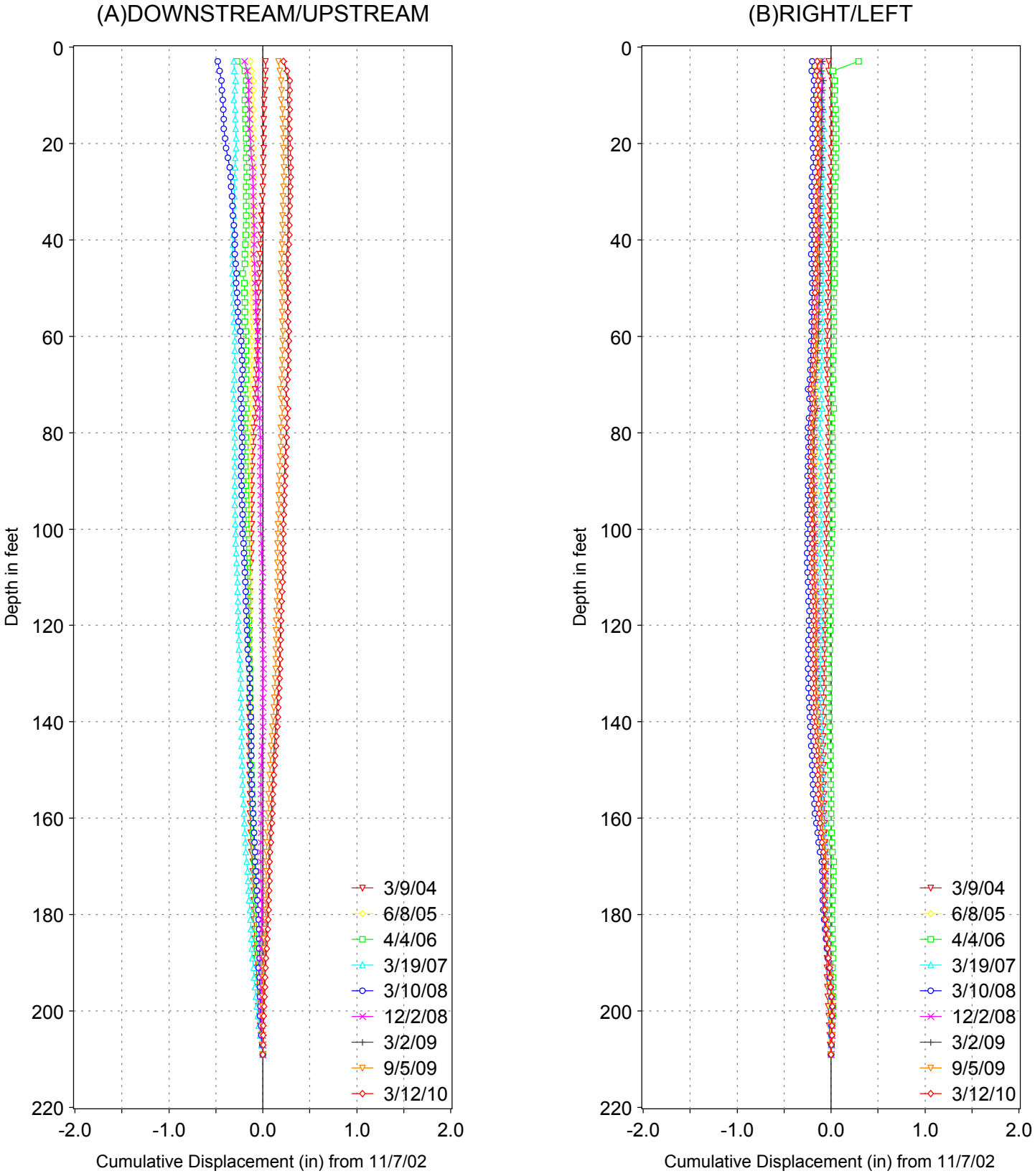
Chatfield Dam and Lake, CO

Inclinometer 526, Outlet Works Sta. 4+36, 9.5' Left

Cumulative Displacement 2006 - Present

Reallocation Study **Plate D-27**



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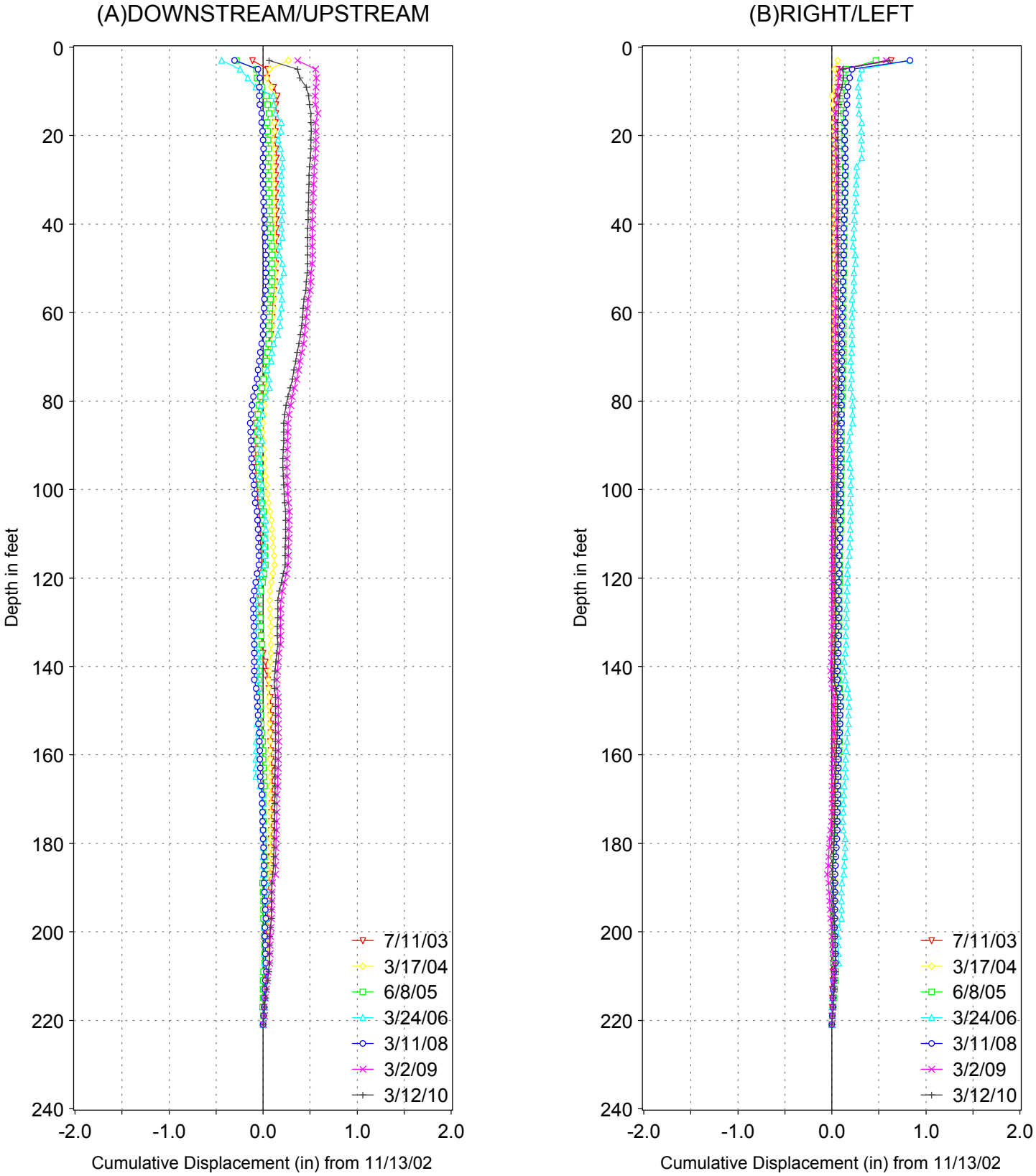
Chatfield Dam and Lake, CO

Inclinometer 544, Sta. 68+80, 17' D.S.

Cumulative Displacement 2002 - Present

Reallocation Study **Plate D-28**



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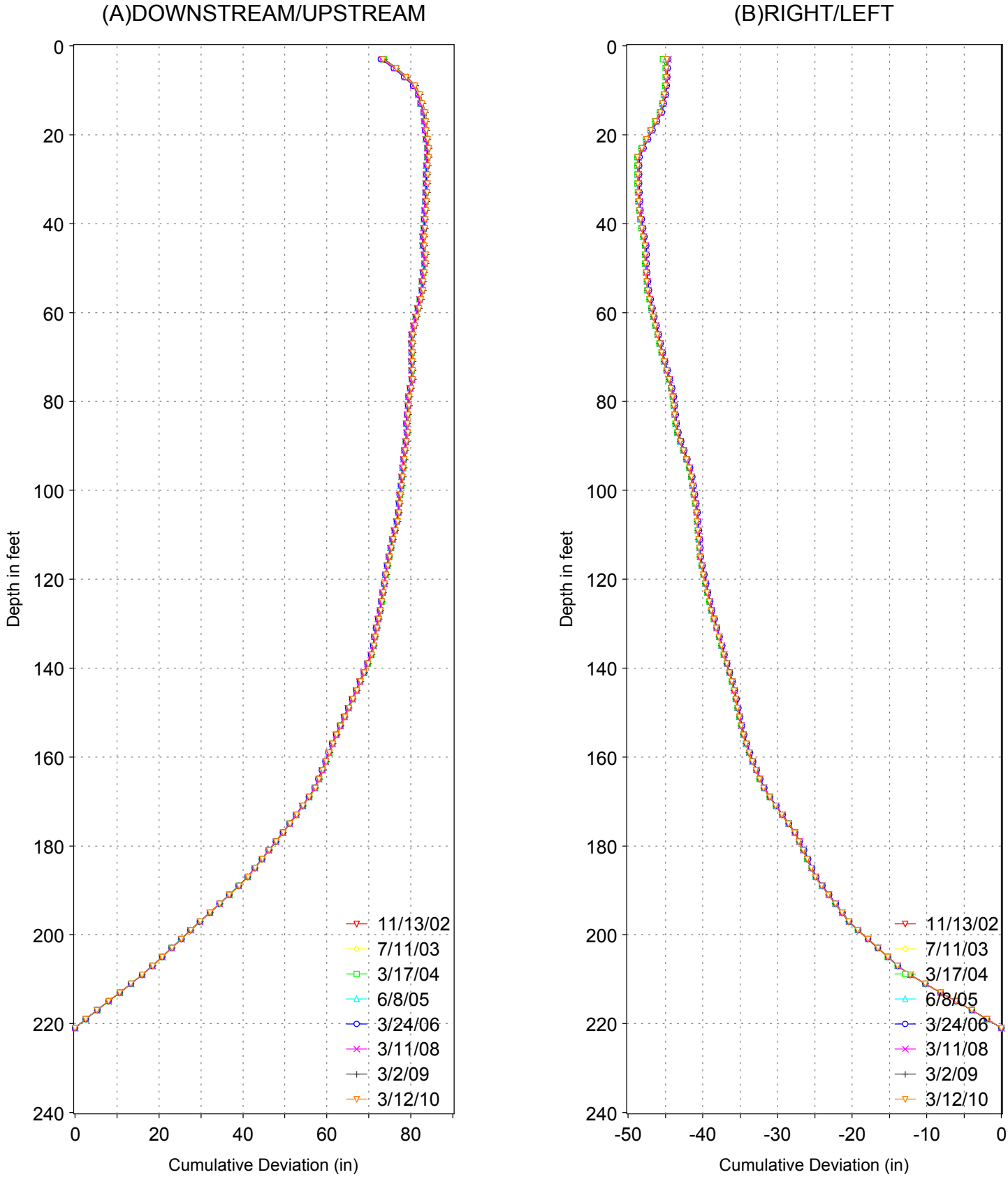
Chatfield Dam and Lake, CO

Inclinometer 545, Sta. 81+10, 17' DS

Cumulative Displacement 2002 - Present

Reallocation Study **Plate D-29**



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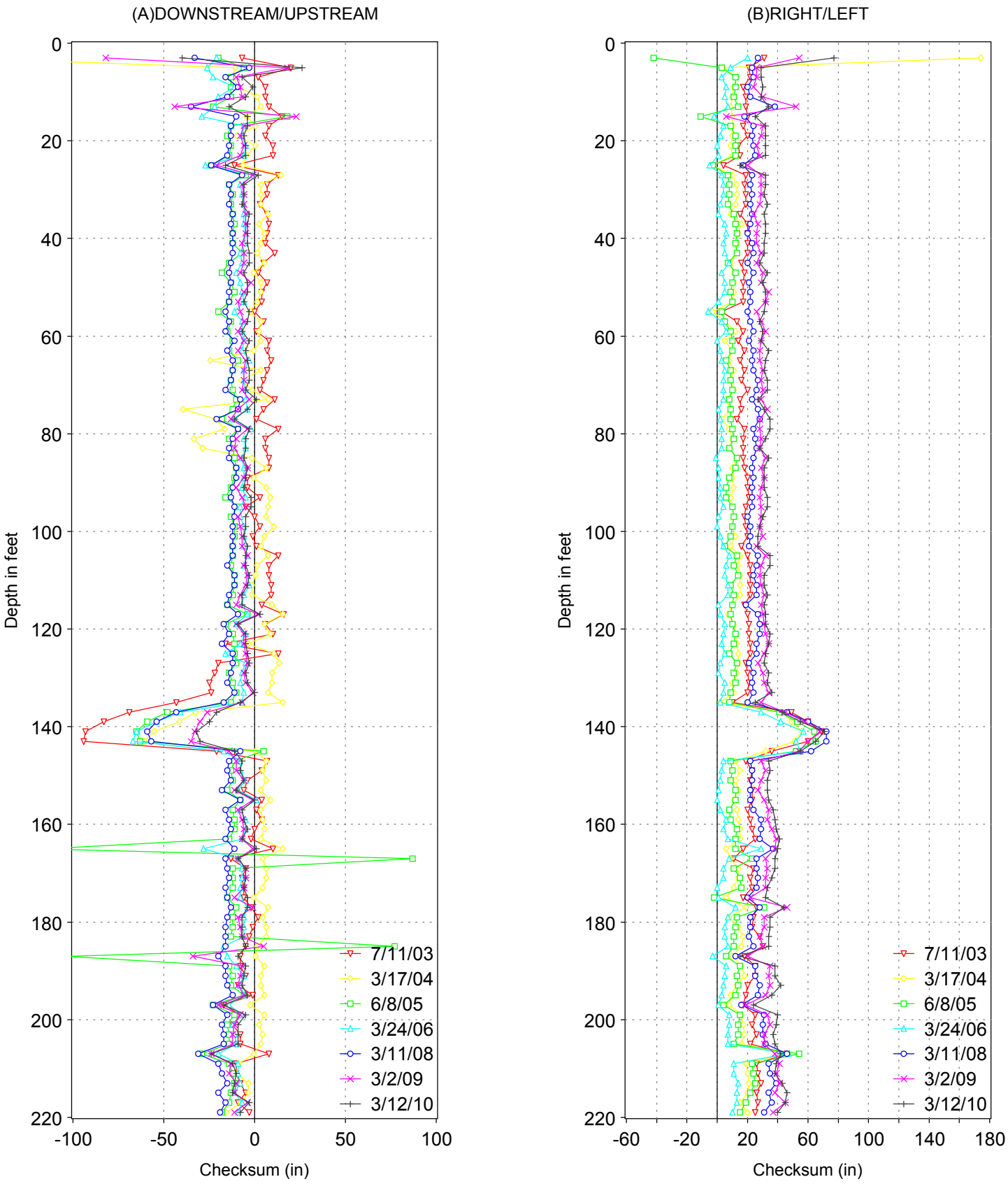
Chatfield Dam and Lake, Co

Inclinometer 545, Embankment Sta. 81+10, 17' DS

Cumulative Deviation 2002 - Present

Reallocation Study **Plate D-30**



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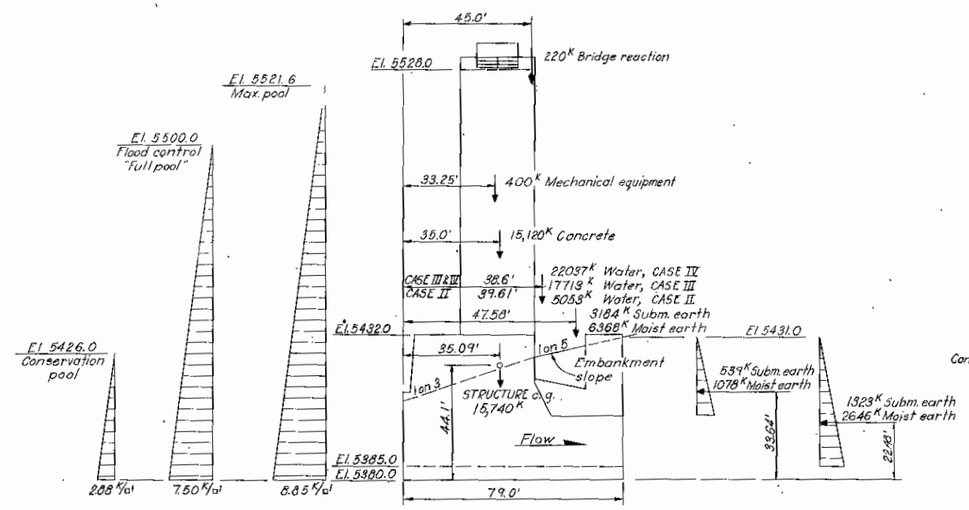
Chatfield Dam and LAke,CO

Inclinometer 545, Sta. 81+10, 17' DS

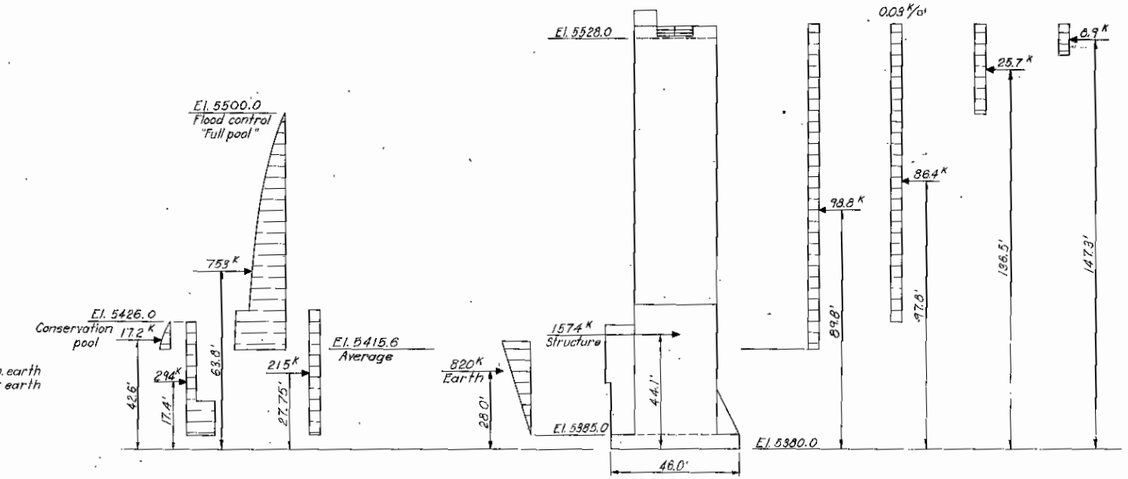
Checksum 2002 - Present

Reallocation Study **Plate D-31**



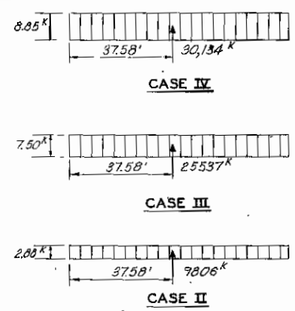
HORIZONTAL WATER PRESSURES

HORIZONTAL EARTH PRESSURES

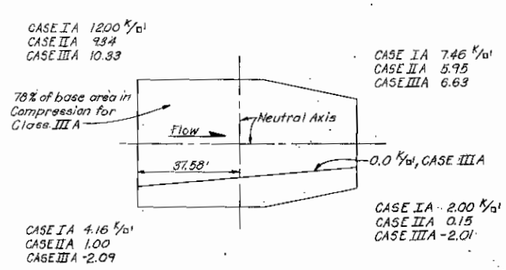


EARTHQUAKE FORCES

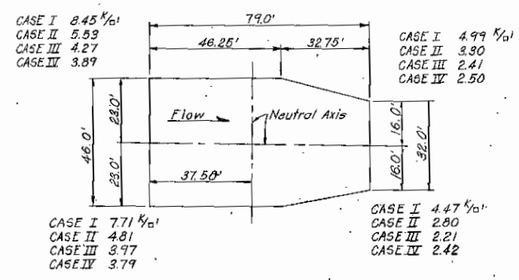
WIND PRESSURES



UPLIFT PRESSURES



FOUNDATION PRESSURES DUE TO EARTHQUAKE



FOUNDATION PRESSURES DUE TO WIND

LOADING CONDITIONS:

CASE I CONSTRUCTION CONDITION
- a. Reservoir empty
- b. Dead load of structure, incl. bridge & equip.
- c. Earth loads
- d. Wind load on side above backfill

CASE II NORMAL OPERATING CONDITIONS
- a. Reservoir at El. 5426, Conservation Pool
- b. All gates open
- c. Dead load of structure
- d. Earth loads
- e. Wind load on side above water surface
- f. Full uplift on base

CASE III FULL FLOOD CONDITION
- a. Reservoir at El. 5500, Full Pool
- b. Emergency gates closed
- c. Dead load of structure
- d. Earth loads
- e. Wind load above water surface
- f. Full uplift on base

CASE IV MAXIMUM FLOOD CONDITION
- Same as Case III except pool at El. 5521.6

Case IA, IIA, OR IIIA
- Same as Case I, II, or III, except earthquake is substituted for wind.

SHEAR-FRICTION FORMULA:

Sliding safety factor = $\frac{fV + sA}{H}$

CASE I	7.56	CASE IA	6.36
CASE II	13.64	CASE IIA	7.73
CASE III	13.06	CASE IIIA	6.33
CASE IV	13.00		

Allowable bearing pressure = 12.0 k.s.f.

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION	MADE	APPRD
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: R. W. A.
DRAWN BY: E. J. G.
TRACED BY:
CHECKED BY: O. F. S.
SUBMITTED BY: Chas. E. Drake, CHIEF STRUCT. SECTION
APPROVED: CHIEF CIVIL DES. BRANCH
APPROVED: Charles L. Shipp, CHIEF ENGINEERING DIVISION
APPROVED: COL. U. S. E. DISTRICT ENGINEER

SOUTH PLATTE RIVER, COLORADO
CHATFIELD DAM AND RESERVOIR
OUTLET WORKS
INTAKE TOWER STABILITY ANALYSES

DATE: JUNE 1967
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

CHATFIELD DAM AND LAKE
SOUTH PLATTE RIVER BASIN
LITTLETON, COLORADO

DAM SAFETY EVALUATION REPORT

PROJECT MANAGEMENT PLAN

1. PRODUCT DEFINITION

It has been proposed to raise the normal reservoir elevation of the Chatfield Reservoir by up to 12 feet, from El. 5432 Ft. MSL to El. 5444 Ft. MSL, for the purpose of water supply. The water re-allocation study will actually evaluate three options, (1) no raise, (2) a five foot raise, and (3) a twelve foot raise. The final adopted plan will not increase the maximum surcharge reservoir elevation. The historic maximum reservoir elevation at Chatfield Dam is 5447.58. Ft. MSL. The purpose of this report is to evaluate potential dam safety concerns based on a permanent increase in the reservoir elevation. The evaluation is based strictly on static loading; however, historic information on previous seismic evaluations will be discussed. It is vital to address various aspects of design and performance to assure that the proposed modifications do not impact the continued safe operation of the dam and do not pose dam safety concerns.

2. PRODUCT DEVELOPMENT TEAM

The product development team consists of:

Michael T. Kelly, CENWO-ED-GB (Geotechnical Engineer)

Larkin Whistler, CENWO-ED-DF (Structural Engineer)

Eric Laux, CENWO-PM-C (Project Manager)

3. QUALITY CONTROL PROCESS

The quality control process will consist of a peer review for each discipline invoved in the product, a quality control review and an independent technical review. All comments will be resolved to the satisfaction of the Peer Review Team, Quality Control Review Team and the Independent Technical Review Team. Those participating on these teams are the following:

PERR REVIEW TEAM

Bob Worden, CENWO-ED-GB (Geotechnical Engineer)

Lyle Peterson, CENWO-ED-DF (Structural Engineer)

QUALITY CONTROL REVIEW TEAM

Richard Taylor, Chief , CENWO-ED-GB

Bruce Harris, Chief, CENWO-ED-GB

INDEPENDENT TECHNICAL REVIEW TEAM

Joseph Topi, CENWK-EC-GD (Geotechnical Engineer)

4.0 SCHEDULE

The proposed schedule milestone dates for the quality control reviews are given below.

Daft Report Complete by 15 Nov 08

Independent Technical Reviews Complete by 1 Dec 08

Peer Reviews Complete by 1 Jan 09

Quality Control Review Complete by 15 Jan 09

Final Report Complete by 1 Feb 09

5.0 DOCUMENTATION

Documentation of the quality control process will be recorded on the following forms. Reviewers will sign and date the forms after their review is complete.

CHATFIELD DAM AND LAKE
SOUTH PLATTE RIVER BASIN
LITTLETON, COLORADO

DAM SAFETY EVALUATION REPORT
PEER REVIEW CERTIFICATION

The undersigned certify that the Chatfield Dam Safety Evaluation report has been reviewed and that all significant comments generated have been addressed.

DATE

APPROVED BY: _____
BOB WORDEN, P.E. SOILS SECTION B,
GEOTECHNICAL ENGINEERING AND SCIENCES BRANCH,
ENGINEERING DIVISION

APPROVED BY: _____
LYLE PETERSON, P.E. STRUCTURAL/ INTERIOR DESIGN
SECTION, ENGINEERING DIVISION

CHATFIELD DAM AND LAKE
SOUTH PLATTE RIVER BASIN
LITTLETON, COLORADO

DAM SAFETY EVALUATION REPORT
QUALITY CONTROL REVIEW CERTIFICATION

The undersigned certify that Chatfield Dam Safety Evaluation Report has been reviewed and that all significant comments generated by the quality control review team have been addressed.

DATE

APPROVED BY: _____
RICHARD TAYLOR, P.E.. CHIEF, SOILS SECTION B
GEOTECHNICAL ENGINEERING AND SCIENCES BRANCH,
ENGINEERING DIVISION

APPROVED BY: _____
BRUCE HARRIS, P.E. CHIEF, STRUCTURAL/INTERIORS SECT.,
DESIGN BRANCH, ENGINEERING DIVISION

CHATFIELD DAM AND LAKE
SOUTH PLATTE RIVER BASIN
LITTLETON, COLORADO

DAM SAFETY EVALUATION REPORT

INDEPENDANT TECHNICAL REVIEW CERTIFICATION

The undersigned certify that the Chatfield Dam Safety Evaluation Report has been reviewed.

DATE

APPROVED BY: _____
Joseph Topi, CENWK-EC-GC

POST-LIQUEFACTION STABILITY ANALYSES DECEMBER 2010

CHATFIELD DAM & LAKE SOUTH PLATTE RIVER BASIN LITTLETON, COLORADO

U.S. Army Corps
Of Engineers
Omaha District
Final Report

Post-Liquefaction Stability Analyses
Chatfield Dam
Littleton, Colorado

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Appendix A – Excerpts From Embankment Criteria and Performance Report

Appendix B – Determination of Residual Shear Strength

Appendix C – Steady-State Seepage Stability Analyses

Appendix D – Post-Liquefaction Stability Analyses

Post-Liquefaction Stability Analyses
Chatfield Dam
Littleton, Colorado

1.0 Introduction This report presents the results of stability analyses performed on zones of the Chatfield Dam foundation that have been identified as susceptible to liquefaction (CENWO, 2009a).

2.0 Background A liquefaction assessment was performed in April 2009 as part of the water supply reallocation study for Chatfield Dam. The reallocation study is evaluating three options for a reservoir raise including: 1) no raise, 2) a five-feet raise in the multi-purpose pool, and 3) a 12-feet raise in the multi-purpose pool.

The scope of the liquefaction assessment (CENWO, 2009a) included an evaluation of the liquefaction potential of the Chatfield Dam embankment and foundation for both the existing multi-purpose reservoir (El. 5432) and the proposed 12-feet raise (El 5444). Granular soils were evaluated using the "Simplified Seed Method" (Youd et.al., 2001) which defines a Factor of Safety (F.S.) as the Cyclic Resistance Ratio (CRR) divided by the Cyclic Stress Ratio (CSR). The CRR is based on Standard Penetration Test (SPT) blow counts, corrected for fines content and hammer efficiency and normalized to an effective overburden pressure of one ton per square foot $(N_1)_{60}$. For the purpose of the liquefaction assessment, Factors of Safety less than 1.1 were deemed to liquefy.

The liquefaction assessment concluded that:

- The Chatfield Dam embankment would most likely be safe against liquefaction for a 6.0 (Mw) maximum credible earthquake with a PGA of 0.32g.
  Evaluation of the Chatfield Dam foundation indicates zones of liquefaction are likely along the upstream and downstream slope in the valley section.

- Review of soil conditions along the right abutment tends to indicate saturated zones of relatively loose silty sand and sand that may be prone to liquefaction.

- Recommendations included conducting post-earthquake limit equilibrium slope stability analyses to evaluate the stability of the upstream and downstream slopes in the valley section and right abutment section due to the potential for liquefaction of the foundation soils.

3.0 Cases Considered Two cross-sections were selected for post-liquefaction slope stability analyses: a maximum valley section at Station 95+00 and a right abutment section at Station 57+60. For each combination of section and slope, two multi-purpose pool levels were considered: the existing pool level (EL. 5432) and a 12-feet raise (El. 5444).

The maximum valley section was evaluated in the previously discussed liquefaction assessment. Based on the F.S. results at Station 95+00 and adjacent borings, an upstream liquefied zone was identified from El. 5375 to El. 5380 and a downstream liquefied zone was identified from El. 5352 to El. 5361.

The right abutment section was evaluated in the 1986 Seismic Evaluation (CENWO, 1986). The SPT values for this section were taken in holes advanced with a churn drill, so there is some concern that the SPT results were influenced by the drilling operation. However, the values were used only for relative comparisons with adjacent values and to identify low SPT zones. Based on the SPT results at Station 57+60 and adjacent borings, an upstream liquefied zone was identified from El. 5426 to El. 5432 and a downstream liquefied zone was identified from El. 5411 to El. 5432. The residual shear strength was determined from the blow counts of nearby SPT-12, which was evaluated in the liquefaction assessment (CENWO, 2009a).

For each section and pool level, two conditions for the horizontal extent of the potential liquefied zone were assumed: a) a zone extending beneath the entire upstream or downstream slope, and b) a zone extending ±100 feet of the embankment toes. Sketches of these liquefied zones are shown on Figures 1a and 1b. The zone under the entire slope is considered conservative, while the zone under the embankment toes is considered more realistic, given that the overburden confining pressure increases with increasing embankment fill height, which reduces liquefaction potential under the slope.

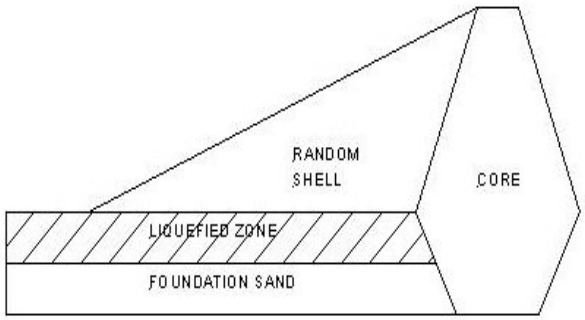

Figure 1a: Liquefied Zone Under Embankment Slope

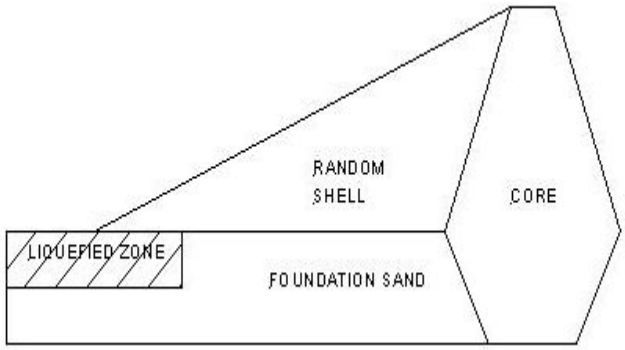

Figure 1b: Liquefied Zone Under Embankment Toe

Prior to performing the post-liquefaction stability analyses, the original design static stability analyses for the steady state seepage condition were reanalyzed using Spencer's method. The embankment geometry, foundation conditions, pool levels, and peak effective shear strengths (CD or S) were obtained from the Embankment Criteria and Performance Report (CENWO, 1980). Excerpts from this report are presented in Appendix A.

4.0 Analytical Approach The analytical approach was based on the guidance provided in the draft EM 1110-2-6001, "Seismic Stability of Earth and Rock Fill Dams," Chapter 8, "Post-Earthquake Stability Analysis" (USACE, 1998). This approach uses the effective stress parameters for the non-liquefied materials and an undrained residual strength for the liquefied zones. Stability analyses are conducted using circular arcs or non-circular surfaces and Spencer's method.

5.0 Adopted Design Parameters

5.1 Embankment Geometry The embankment geometry for the valley section (Sta. 95+00) was obtained from the "Embankment Criteria and Performance Report" (CENWO, 1980). The embankment geometry for the right abutment section (Sta. 57+60) was obtained from the 1986 Seismic Evaluation (CENWO, 1986).

5.2 Embankment Zoning The Chatfield Dam is a rolled, zoned, earthfill embankment. The embankment zoning consist of a symmetrical central impervious core extending to the Dawson formation bedrock, upstream and downstream random material shells, and a downstream pervious inclined sand drain with continuous outlets adjacent to the impervious core. The outer portion of the downstream random zone includes a zone specifically for all Dawson formation materials excavated from the spillway and outlet works excavations. This was done to keep the Dawson formation material least susceptable to saturation. A cross-section of the maximum valley section (station 95+00) is presented in Figure 2.

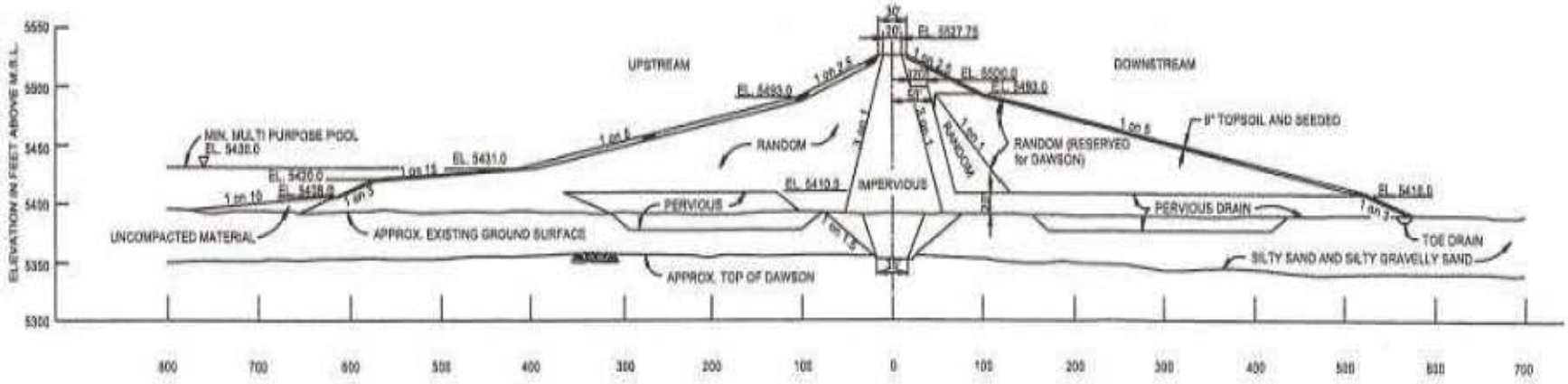

Figure 2: Embankment Zoning and Foundation Conditions In Valley
(Station 75+00 – Station 95+00)

5.3 Foundation The dam foundation consists of sands, gravels, and sandy clay alluvium derived from the weathering and erosion of the parent materials of the mountains to the west, underlain by uncemented sand and sandy gravel and the Dawson formation bedrock. This study focuses on the granular alluvium and overburden soils overlying the Dawson. A cross-section of the maximum valley section is presented in Figure 2.

5.4 Shear Strengths The peak effective stress parameters (CD or S strengths) were selected from the "Embankment Criteria and Performance Report" (CENWO, 1980) for the non-liquefied materials (See Appendix A). These values are presented in Table 1.

Table 1: Adopted Peak Effective Stress (CD or S) Shear Strengths

Material	Cohesion (psf)	Friction angle (degrees)
Embankment		
Impervious	0	24.3
Random	0	24.3
Pervious	0	36.1
Dawson Fm.	0	22.3
Foundation		
Clay	0	20.8
Sand	0	33.0
Dawson Fm.	0	15.0

For the liquefied zones, an undrained residual strength was determined from a plot developed by Seed et.al., as reproduced in a report by Marcuson, Hynes and Franklin (1990). This plot (Figure 3) relates the undrained residual strength (in pounds per square foot) to the corrected, equivalent clean sand, SPT blow counts $(N_1)_{60}$.

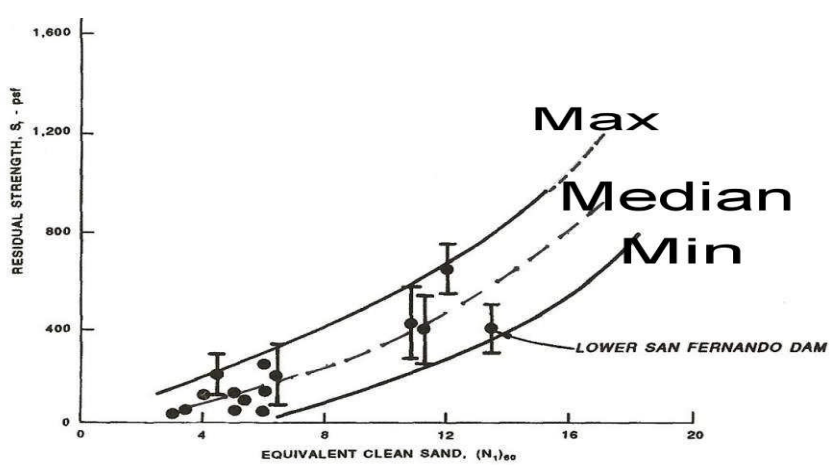

Figure 3: Relationship Between Residual Strength and SPT N-values (Marcuson, Hynes and Franklin, after Seed et.al.)

Corrected SPT values corresponding to Factors of Safety ≤ 1.1 were used to determine the residual shear strength for the liquefied zones. Three residual strength levels were considered:

1. Residual Strength Level 1 (RSL-1). Residual strengths for corrected SPT values were determined from the "Median " line of Figure 3. Individual residual shear strength values were ranked and an adopted design value was selected such that two-thirds (67 percent) of the test values exceed the adopted design value (USACE, 1970).

2. Residual Strength Level 2 (RSL-2). Residual strengths for corrected SPT values were determined from the "Median " line of Figure 3. Individual residual shear strength values were ranked and an adopted design value was selected such that one half (50 percent) of the test values exceed the adopted design value.

3. Residual Strength Level 3 (RSL-3). Residual strengths for corrected SPT values were determined from the "Max" line of Figure 3. Individual residual shear strength values were ranked and an adopted design value was selected such that one half (50 percent) of the test values exceed the adopted design value.

The corrected SPT values, selected residual strengths, and calculation for the adopted design residual strength for each strength level are presented in Appendix B. The adopted residual shear strengths are summarized in Table 2.

Table 2: Adopted Undrained Residual Shear Strengths

Location	**Residual Strength, psf**		
	RSL-1	**RSL-2**	**RSL-3**
Station 95+00			
Upstream	470	480	680
Downstream	540	560	790
Station 57+60			
Upstream	760	880	1120
Downstream	760	880	1120

5.5 Phreatic Surfaces The phreatic surfaces for the maximum valley section (Station 95+00) were obtained from the Liquefaction Assessment (CENWO, 2009). The phreatic surface for the right abutment section (Station 57+60) was obtained from the Seismic Evaluation (CENWO, 1986).

6.0 Results Non-circular, block failure surfaces were analyzed for each upstream or downstream stability case with Spencer's method, utilizing the 2007 version of SLOPE/W, developed by Geo-Slope International, Ltd.

For the re-analysis of the steady state seepage cases, the critical slide plane and peak effective strengths were used, as presented in the Embankment Criteria and Performance Report (CENWO, 1980). Excerpts from this report are presented in Appendix A. The intent of these re-analyses was to compare the current methodology (Spencer's method) with the method used for the original design.

For each post-liquefaction stability case, the sliding elevation was assumed horizontal and the location of the active and passive wedges were iterated until the coordinates corresponding to the critical F.S. were bounded by higher F.S. values. A minimum value of 1.30 was adopted for the post liquefaction factor of safety (CENWO, 2009a).

6.1 Steady State Seepage Cases Two embankment sections were considered in the original design, i.e., Station 95+00 and Station 68+50. Station 95+00 is typical of the valley section from Station 75+00 to Station 95+00. The embankment attains a height of about 117 feet and the foundation sand and clay are about 55 feet thick. Station 68+50 is typical of the right valley section. The embankment is about 131 feet high, with the Dawson Formation at the ground surface under the downstream slope and 30 to 40 feet below the ground surface under the upstream slope.

The results of the re-analysis of the steady seepage cases at Station 95+00 and Station 68+50 are presented on Table 3. These results indicate the factors of safety determined with Spencer's method exceed those factors of safety determined during the original design. Figures C-1 and C-2 in Appendix C present a comparison of the original stability analysis geometry and loading with the graphical output from the Slope/W re-analysis of each case.

Table 3: Steady State Seepage Factors of Safety

Station	Factor of Safety		
	Original	Re-Analysis	
		Janbu's Method	Spencer's Method
95+00	1.43	1.46	1.74
68+50	1.62	1.57	1.84

The difference in the Factor of Safety between the original analyses and the re-analyses using Spencer's method is due to the side force assumptions and statics of each method. The USACE method used in the original design analyses assumes a side force orientation and solves only for horizontal force equilibrium. Spencer's method iteratively determines the side force orientation and is statically determinate. Janbu's Generalized method, which makes assumptions similar to the USACE method, was used to analyze the same Slope/W model. The results, shown in Table 3, substantiate the original design analyses and confirm the difference in factor of safety is due to the analytical method.

6.2 Post-Liquefaction Cases

6.2.1 Station 95+00 The stability analysis results for the maximum valley section are summarized on Table 4. The upstream embankment factors of safety range from 1.18 to 2.38, depending on conditions and assumptions utilized. The downstream embankment factors of safety range from 1.17 to 1.85, depending on conditions and assumptions utilized.

Table 4: Post Earthquake Factors of Safety, Station 95+00

Liquefied Zone Extent	Residual Strength		
	RSL-1	RSL-2	RSL-3
Pool El. 5432			
Upstream Slope	1.18	1.19	1.40
" Toe	1.72	1.74	2.15
Downstream Slope	1.18	1.19	1.33
" Toe	1.68	1.70	1.85
Pool El 5444			
Upstream Slope	1.24	1.25	1.44
" Toe	1.92	1.95	2.38
Downstream Slope	1.17	1.18	1.32
" Toe	1.66	1.67	1.83*

* Further details of this case are presented in Appendix D.

The case representing pool El. 5444, with a liquefied zone limited to the downstream toe and the RSL-3 residual shear strength (F.S. = 1.83) is considered representative of the post-earthquake stability at this station since the overburden confining stress is lowest at the embankment toe. This case is presented on Figure D-1 in Appendix D.

6.3.1 Station 57+60 The stability analysis results for the right abutment section are summarized on Table 5. The upstream embankment factors of safety range from 1.15 to 2.75, depending on conditions and assumptions utilized. The downstream embankment factors of safety range from 1.58 to 3.75, depending on conditions and assumptions utilized.

Table 5: Post Earthquake Factors of Safety, Station 57+60

Liquefied Zone Extent	Residual Strength		
	RSL-1	RSL-2	RSL-3
Pool El. 5432			
Upstream Slope	1.16	1.26	1.44
" Toe	2.18	2.18	2.75
Downstream Slope	1.58	1.67	1.84
" Toe	3.68	3.70	3.75

Pool El 5444			
Upstream Slope	1.15	1.24	1.42
" Toe	2.10	2.29	2.68*
Downstream Slope	1.58	1.67	1.84
" Toe	3.62	3.64	3.69

* Further details of this case are presented in Appendix D.

The case representing pool El. 5444, with a liquefied zone limited to the upstream toe and the RSL-3 residual shear strength (F.S. = 2.68) is considered representative of the post earthquake stability at this station since the overburden confining stress is lowest at the embankment toe. This case is presented on Figure D-2 in Appendix D.

7.0 Conclusions

The results of this study suggest the following conclusions:

- Cases representing the liquefied zone at the toes of the upstream and downstream slopes are considered the most representative of the field conditions.

- The conservative cases combining the lowest residual strength (RSL-1) and a liquefied zone extending under the entire slope length have a F.S. of nominally 1.2.

- All F.S. determined using the RSL-3 residual strength exceed the minimum F.S. of 1.3, regardless of the horizontal extent of the liquefied zone.

- The combination of the toe liquefied zones and the RSL-3 residual strength best represent the liquefaction potential of the Chatfield Dam foundation. The minimum Factor of Safety for these cases is 1.83 for the Station 95+00 section and 2.68 for the Station 57+60 section.

- None of the critical failure surfaces breach the embankment crest.

8.0 References

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Youd, T.L., et.al, (2001). "Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 127, No. 10, pp 817-833.

APPENDIX A

EXCERPTS FROM EMBANKMENT CRITERIA AND PERFORMANCE REPORT

SOUTH PLATTE RIVER BASIN

CHATFIELD LAKE, COLORADO

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

United States Army
Corps of Engineers
...Serving the Army
...Serving the Nation

Omaha District

APRIL 1980

3.11.2.6. Seepage Under or Around Cutoff. Investigations were made to determine whether seepage could be a problem either under or around the cutoff trench. It was found that any seepage that might occur under the cutoff in the sandstones of around the end of the cutoff in the right abutment would be of such a small quantity that it would not be particularly noticeable. Piezometers were installed in the sandstones of the Dawson Formation near the downstream toe of the embankment to monitor seepage pressures that occur in the sandstones.

3.11.2.7. Underground Water Rights. In studies concerning the positive cutoff trench extending to bedrock, it was assumed that the groundwater table immediately downstream of the dam would be adversely affected and may produce serious problems for downstream holders of underground water rights. In connection with this, the Colorado Water Conservation Board was asked to assist in determining the criteria that should be considered for underground water requirements in the area. They concluded that with one or two possible exceptions, an impervious cutoff would have no noticeable effect on the majority of wells in existence and that even if some effect could be demonstrated on the remainder of the wells, the well owners could not prove a legal injury since they were illegally diverting water from the South Platte River. Their conclusion was based on a study area of five miles downstream from the dam, which included 35 wells on record, 18 of which were able to be actually located in the field. Their study indicated that most of the wells were located a distance of a mile or more from the dam site and were supplied largely from irrigation ditches.

3.12. Design Shear Testing.

3.12.1. Remolded Testing. Shear tests were conducted on remolded samples of materials which were representative of materials to be encountered in the required excavations and borrow areas. The

remolded samples were tested at 95 percent of maximum density, as determined by AASHO T-99-57A, at optimum and optimum plus 3 percent moisture content. A single test usually consisted of three specimens tested at different confining stresses. A total of 16 unconsolidated undrained "Q" tests; 20 consolidated-undrained "R" tests; and 18 consolidated-drained "S" tests were performed. Samples for "R" tests were saturated by back pressure before shearing. Triaxial specimens were 1.4 inch diameter except for a few at 3.9 inch diameter for coarse grained material. Direct shear tests were 3 in. by 3 in. by 0.5 in. and were sheared about 9.5 inch at a rate of about 0.0038 inch per minute. Test summaries showing adopted strengths are shown on Plates B-55 thru B-57.

3.12.2. Undisturbed Testing. Triaxial ("Q" and "R") and direct shear ("S") tests were performed on selected undisturbed samples of the clays in the valley foundation, and abutments plus the Dawson formation of bedrock. A single test usually consisted of 3 specimens tested at different confining stresses. Size of specimens were similar to that performed for remolded testing. Summaries of the foundation soils testing are shown on Plates B-58 thru B-60, and summaries of the shear tests on the Dawson Formation are on Plates B-61 thru B-69.

The total number of undisturbed tests conducted are as follows:

No. of Undisturbed Tests

Type of Test	No. of Shear Tests Foundation Soil	Foundation Bedrock
Q	18	11
R	15	4
S	14	19

3.12.3. Residual Testing. A total of 28 direct shear residual tests were performed on undisturbed Denison samples and on specimens obtained from cubic foot box samples of the Dawson Formation materials. Discussion and results are given in paragraph 3.12.4.3. The test summaries are shown on Plates B-70 and B-71.

3.12.4. Test Results.

3.12.4.1. Embankment Materials. Most of the remolded testing was concentrated on the weaker materials from each area of excavation. The impervious and random materials were chosen to have identical strengths and the final adopted strength was based primarily on the lowest strength of all the material types tested. Additionally, since the majority of the material would come from excavations where the moisture content was from 2 to 6 percent dry of optimum, the final adopted shear strengths were based on optimum moisture content specimen testing.

It was determined that "Q" tests conducted at optimum moisture were about twice the strength of those conducted at optimum plus 3 percent.

3.12.4.2. Foundation Soils. The majority of tests performed on foundation materials were on the weaker surface clays in the valley. Results indicated material of relatively low shear strength which ultimately was required to be removed to satisfy embankment stability requirements.

Results of testing indicated considerable ranges in shear strength which gave evidence to the heterogeneous nature of the foundation soils in general. Adopted strengths were based on the lowest test strengths obtained.

3.12.4.3. Bedrock. Tests on the Dawson Formation materials was generally confined to the clay-shale material which was thought to be weaker than the silty or sandy shales, sandstones and siltstones. Specific concentration was made of the more weathered bedrock. Additionally, since the Dawson Formation contains numerous slickensides of all sizes and degrees, the testing and correlation concerned this effect upon the strength. Results of the tests indicated that the slickensides in the bedrock appeared to have little

effect on the strength while the affects of weathering appeared to give lower strengths.

Stress-strain curves on direct-shear tests of the Dawson Formation exhibited sharp peaks and substantial differences between peak strengths and strength at the end of the strain, commonly referred to as ultimate strength. Because of the relatively sharp drop in strength with strain and the slickensided nature of the Formation, it was thought that slight additional amounts of horizontal movement may lower the strength toward a residual strength which could be considerably lower than the ultimate strength. As a consequence, a residual testing program was developed. Tests were performed on precut samples both by MRD and SWD laboratories in direct shear boxes having a size of 3"x3"x1". Total displacements ranged from 5 to 11.6 inches with residual strength usually being reached at from 4.4 to 11 inches. The range in residual strengths ranged from $\phi = 5°$ to $\phi = 32°$.

The final adopted strength of the Dawson Formation "S" strength was a practical value selected as a reasonable and conservative strength for use in stability computations. The value $\phi = 15°$ is approximately midway between a peak value of $\phi = 24°$ and a residual value of $\phi = 8°$.

A range of direct shear strengths by type of material for the Dawson Formation is given below and is also shown on Plate B-69:

Type of Material	Range of Direct Shear Strength	
	Peak	Ultimate
Clay shale	19° to 27°	12° to 23°
Silty shale & Siltstone	27° to 38°	23° to 33°
Sandy shale & Sandstone	38° to 42°	33° to 37°

A summary of residual shear strength test data and mineral composition of the Dawson Formation are shown on Plate B-71.

3.12.5. Summary of Adopted Design Strengths. The adopted shear strengths of the embankment and foundation materials which have been used in stability analyses computations are as follows:

Material	Unconsolidated-Undrained "Q" Strength			Consolidated-Undrained "R" Strength			Consolidated Drained "S" Strength		
	tan Ø	Ø	coh-T/SF	tan Ø	Ø	coh-T/SF	tan Ø	Ø	coh-T/SF
Embankment									
Impervious	0.042	2.4°	1.5	0.17	9.7°	0.50	0.45	24.3°	0.0
Random	0.042	2.4°	1.5	0.17	9.7°	0.50	0.45	24.3°	0.0
Pervious	0.73	36.1°	0.0	0.73	36.1°	0.00	0.73	36.1°	0.0
Dawson Fm.	0.00	0.0°	2.0	0.20	11.3°	0.40	0.41	22.3°	0.0
Foundation									
Clay	0.00	0.0°	0.28	0.15	8.5°	0.30	0.38	20.8°	0.0
Sand	0.65	33.0°	0.00	0.65	33.0°	0.00	0.65	33.0°	0.0
Dawson Fm.	0.00	0.0°	2.70	0.35	19.3°	0.40	0.27*	15.0°*	0.0*

*Value used in lieu of "S" strength.

3.12.6. Additional Shear Testing of Dawson Fm. A post-design testing program was done for soft seam materials of the Dawson Formation. The results of the testing was reported in a Supplement to Design Memorandum No. PC-24, December 1970. The testing was a result of concerns voiced during a Board of Consultants meeting and the objective was to ascertain whether shear strengths of soft seam material might be lower than shear strengths previously obtained for the Dawson Formation. Testing was also done to investigate consolidation and mineralogical characteristics of the seam. Samples were obtained from box samples which were cut from test pits 8 and 9 located in the outlet works area. In addition, some specimens were cut directly into shear boxes. Photos 8 and 9, Plate A-8, show typical undisturbed sampling operations.

The testing program consisted of twelve (12) residual shear tests; and eleven (11) direct shear tests on the soft seam material. Only three of the tests were "precut" residuals similar to that done previously for the Dawson Formation.

Results of the tests indicated that both the residual and normal direct shear strengths of the soft seam materials were within the range of their respective strengths previously obtained for the Dawson Formation. Plate B-72 shows summary results of the testing.

Atterberg Limits and moisture content tests of the soft seam material was compared to the material above and below the seam and the following was found:

(1) The moisture content of the seam material was slightly higher than the surrounding shale.
(2) There was no appreciable difference between Atterberg Limits for the seam material and surrounding shale.
(3) Moisture contents of the seam material exceeded the plastic limit more consistently than the weathered shale or the shale above and below the seam.

Mineralogical tests on the shale and soft seam material indicated that percentages of various clay minerals in the seam material do not differ significantly from that found in the shale material above and below the seam. The dominant absorbed ion was the calcium ion which was also found to be dominant in previous tests on clay shale materials. The results of the mineralogical tests were highly supportive of the conclusion that the seam material is a gouge or fracture type of material and not a separately deposited material, such as bentonite.

3.13. Record Shear Tests. Twenty two undisturbed cubic-foot box samples were taken on the embankment materials by the Stage III Earthwork Contractor during the contract period. Plates C-1 thru C-6,

Appendix "C", show the locations of the box samples and the test results compared to design shear strengths.

The total number of record shear test series on the different materials are presented below:

No. of Record Shear Tests

Type of Test	Embankment Materials Imp & Random	Dawson Fm.
"Q"	12	3
"R"	9	2
"S"	9	2

3.14. Embankment Stability Analyses.

3.14.1. General. Stability analyses were performed for three embankment sections: (1) embankment section at station 95+00 where the embankment attains a maximum height of 137 feet and the depth of the alluvial material is about 55 feet. (2) The outlet works section, where the embankment attains a height of about 117 and alluvial material is 25 feet deep; and (3) the right valley embankment section at station 68+50 where the embankment is about 131 feet high but where the Dawson Formation is at the ground surface for the downstream portion of the section and 30 to 40 feet below the surface under the upstream portion of the section. Analyses consisted of four types of cases which simulate conditions of stress during the life of the structure. The cases are: (1) end of construction; (2) sudden drawdown; (3) partial pool and (4) steady seepage.

3.14.2. Method of Analysis. The sliding wedge method was used for the stability analyses. The factor of safety in the analyses is defined as the ratio of the available shear strength to the average necessary to maintain equilibrium. Most of the studies were performed with an RCA 301 computer using slope stability program 41-R3-1302C, "Slope Stability, Wedge Method."

Most of the analyses were performed in the conventional manner; however, some special cases were performed using at rest pressure conditions for driving forces when failure was assumed in the Dawson Formation. This was done as a means to account for strain incompatibility between the brittle Dawson Formation and the embankment materials. The earth pressure coefficient used for computing the driving forces was 0.5.

3.14.3. Seismic Coefficient. The stability analyses include an allowance for earthquake forces for all potential failure surfaces and cases studied except sudden drawdown from maximum pool. This was done by the addition of a horizontally directed static force in the computations with no change in strengths. The additional earthquake force is the product of a seismic coefficient and the weight of the sliding mass. The coefficient assumed for the analyses was 0.1.

3.14.4. Summary of Results. The following table summarizes the stability analyses results and Plates B-73 thru B-78 summarize each analyses.

CASE	ELEV. OF FAILURE PLANE	NORMAL FACTOR OF SAFETY	REQUIRED FACTOR OF SAFETY	EARTHQUAKE FACTOR OF SAFETY	REQUIRED EARTHQUAKE FACTOR OF SAFETY	SHEAR STRENGTH
END OF CONSTRUCTION						
Upstream-Sta 95+00	5410	2.62	1.3	1.82	1.0	"Q"
Downstream-Sta 95+00	5335	2.53	1.3	1.62	1.0	"Q"
PARTIAL POOL						
Sta 95+00	5410	1.49	1.5	1.04	1.0	"R"
Sta 104+35	5353	1.46	1.5	0.90	1.0	*
SUDDEN DRAWDOWN						
Max. Pool-Sta 95+00	5410	1.23	1.0	-	Not Req'd	"R"
Spillway Pool-Sta 95+00	5410	1.33	1.20	0.92	1.0	"R"
STEADY SEEPAGE						
Sta 95+00 (Conventional)	5300	1.43	1.5	0.86	1.0	*
Sta 95+00 (At-rest pressure)	5335	1.13	-	-	-	*
Sta 104+35(Conventional)	5320	1.58	1.5	0.92	1.0	*
Sta 68+50	5320	1.62	1.5	0.94	1.0	*

* The "S" strength was used for all materials except the Dawson Formation. A lower strength was adopted for the Dawson Formation in lieu of an "S" Strength.

APPENDIX B

DETERMINATION OF RESIDUAL SHEAR STRENGTH

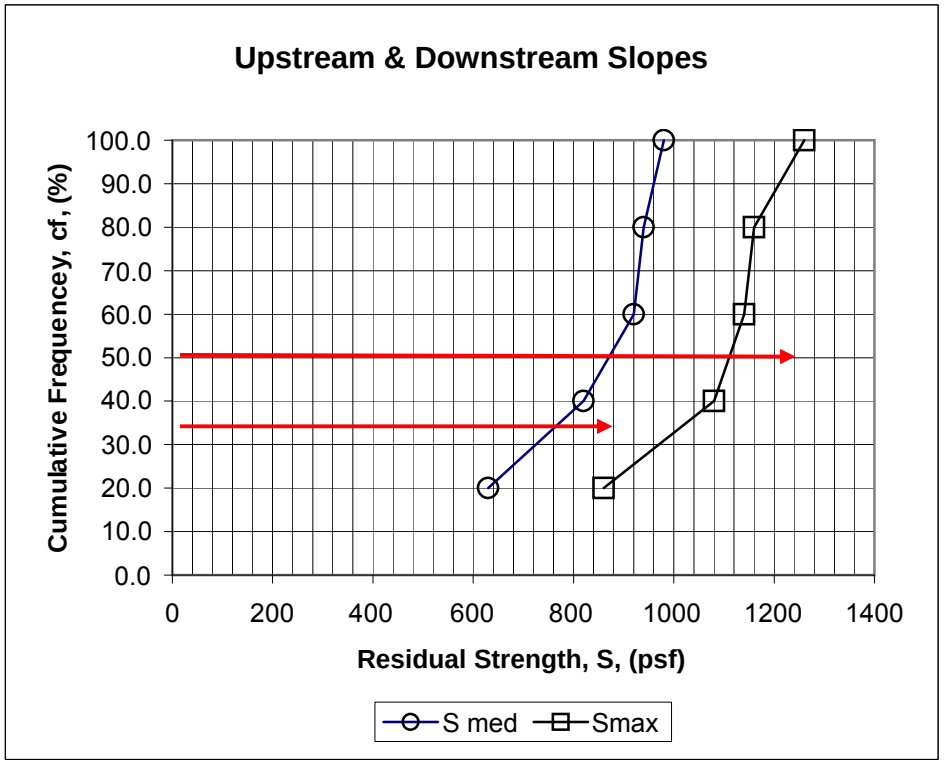
CHATFFIELD DAM
POST-LIQUEFACTION SHEAR STRENGTH

Station 57+60					
Upstream & Downstream					
$(N1)_{60}$ (bpf)	S_{med} (psf)	S_{max} (psf)	n (-)	$\sum n$ (-)	cf (%)
14.0	630	860	1	1	20.0
16.3	820	1080	1	2	40.0
16.5	920	1140	1	3	60.0
16.6	940	1160	1	4	80.0
17.6	980	1260	1	5	100.0

Notes:

1. $(N1)_{60}$ data from Boring SPT-12 for F.S. < 1.1
2. S values obtained from Seed et al. plot

Adopted Residual Shear Strength, (psf)			
Location	**RSL-1**	**RSL-2**	**RSL-3**
Upstream	760	880	1120
Downstream	760	880	1120

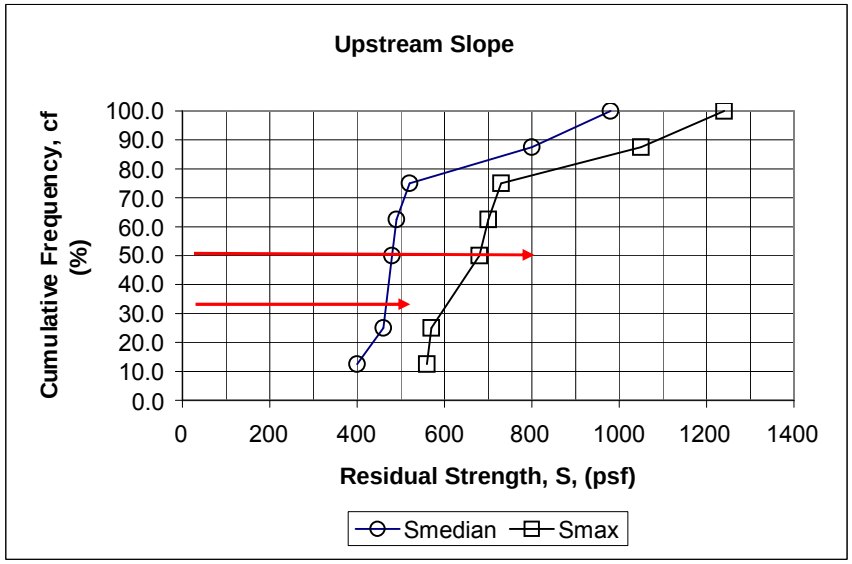


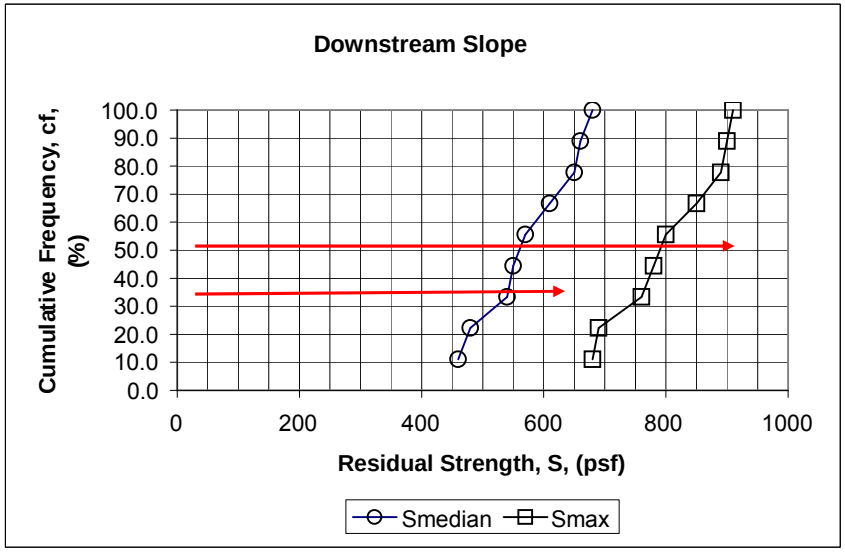
CHATFFIELD DAM
POST-LIQUEFACTION SHEAR STRENGTH

Station 95+00					
Upstream					
$(N1)_{60}$	S_{median}	S_{max}	n	∑n	cf
(bpf)	(psf)	(psf)	(-)	(-)	(%)
10.1	400	560	1	1	12.5
10.3	460	570	1	2	25.0
11.8	480	680	2	4	50.0
12.0	490	700	1	5	62.5
12.4	520	730	1	6	75.0
16.0	800	1050	1	7	87.5
17.6	980	1240	1	8	100.0
Downstream					
11.7	460	680	1	1	11.1
11.8	480	690	1	2	22.2
12.8	540	760	1	3	33.3
13.0	550	780	1	4	44.4
13.2	570	800	1	5	55.6
13.8	610	850	1	6	66.7
14.3	650	890	1	7	77.8
14.4	660	900	1	8	88.9
14.6	680	910	1	9	100.0

Notes:

1 Upstream $(N1)_{60}$ data from Borings SPT-10 & 14 for F.S. < 1.1
2 Downstream $(N1)_{60}$ data from Boring SPT 08-03 for F.S. < 1.1
3 S values obtained from Seed et.al. plot





Adopted Residual Shear Strength. (psf)			
Location	**RSL-1**	**RSL-2**	**RSL-3**
Upstream	470	480	680
Downstream	540	560	790

APPENDIX C

STEADY-STATE SEEPAGE STABILITY ANALYSES

CHATFIELD DAM
Steady-Seepage Stability Re-Analysis

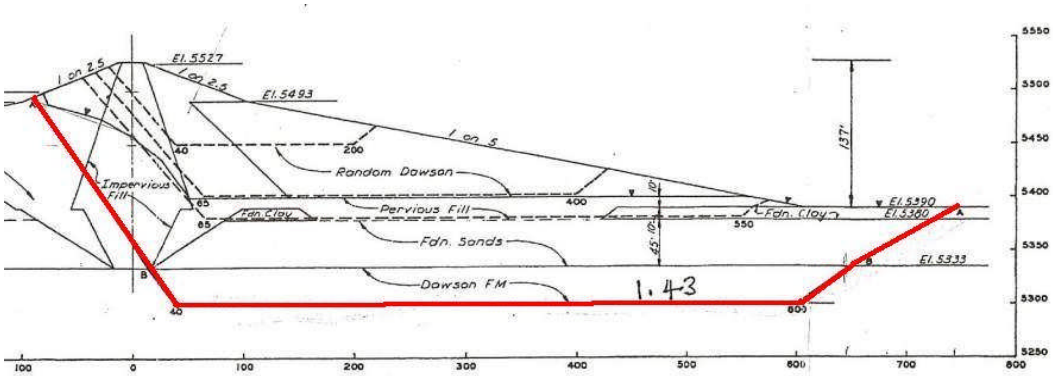


Figure C-1a: Original Stability Analysis from "Embankment Criteria and Performance Report," April, 1980.

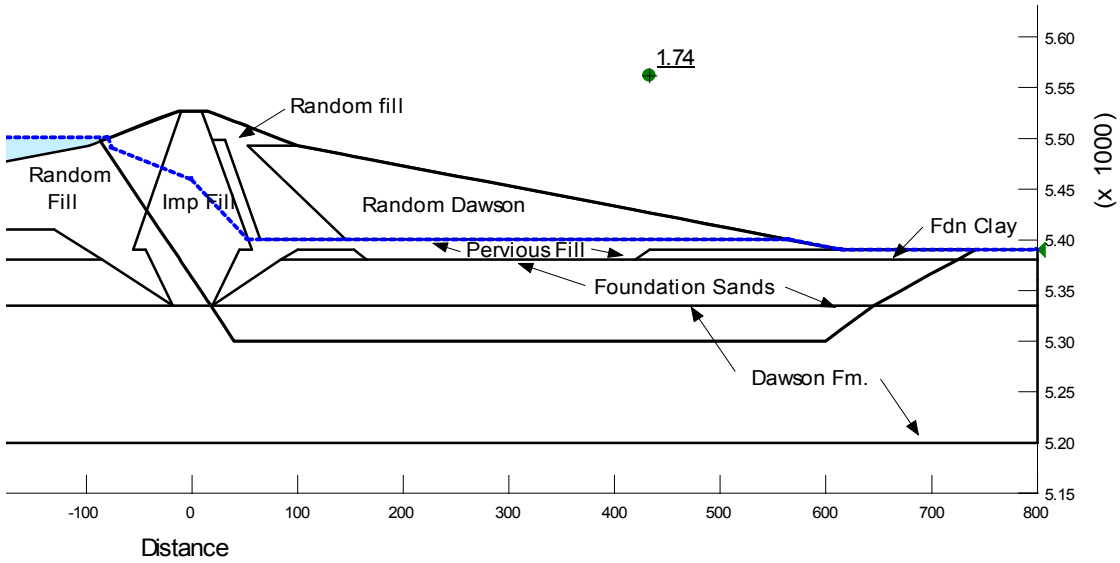


Figure C-1b: Re-Analysis Output from Slope/W

Figure C-1: Steady-Seepage Re-Analysis, Station 95+00

Station 95+00, Steady Seepage, Pool 5500

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File Information

Title: Chatfield Dam, Post Liquefaction Stability Assessment
Comments:
Created By: Stacey, Teryl L NWO
Revision Number: 45
Last Edited By: R.L.Donovan
Date: 09/04/2009
Time: 10:32:50 AM
File Name: Chatfield Sta 95 Steady Seepage.gsz
Directory: F:\Projects\Other\Chatfield Dam\
Last Solved Date: 09/04/2009
Last Solved Time: 10:32:56 AM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Station 95+00 Pool 5500

Description: See Plate B-73 for referenced data.
Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

SlipSurface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Fully-Specified

Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
FOS Distribution
FOS Calculation Option: Constant
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Embankment Impervious

Model: Bilinear
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi 1: 24.3 °
Phi 2: 17.2 °
Bilinear Normal: 3500 psf
Pore Water Pressure
Piezometric Line: 1

Embankment Dawson

Model: Bilinear
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi 1: 22.3 °
Phi 2: 17 °
Bilinear Normal: 3700 psf
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Embankment Random

Model: Bilinear
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi 1: 24.3 °
Phi 2: 17.2 °
Bilinear Normal: 3500 psf

Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Embankment Pervious

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 36.1 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Clay

Model: Bilinear
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi 1: 20.8 °
Phi 2: 14.8 °
Bilinear Normal: 2500 psf
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Sand

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Dawson

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 15 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Limits

Left Coordinate: (-800, 5390) ft
Right Coordinate: (800, 5390.0789) ft

Fully Specified Slip Surfaces

Fully Specified Slip Surface 1

	X (ft)	Y (ft)
	-88	5499
	18	5335
	40	5300
	600	5300
	645	5335
	745	5392

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	-800	5499.9853
	-79.5642	5500.1743
	-75.8814	5490.203
	0.3135	5460.2447
	52.5913	5400.1504
	565	5400
	615	5390
	800	5390.0789

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	1	1.70	(337.761, 5524.05)	378.06	(-86.8702, 5497.25)	(741.586, 5390.05)

Slices of Slip Surface: 1

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)

1	1	-83.21718	5491.6	534.96891	726.42301	86.444845	0
2	1	-77.7228	5483.099	754.36791	1540.7205	355.05183	0
3	1	-59.36778	5454.701	1810.2308	4522.2899	1224.5416	0
4	1	-35.72581	5418.123	3512.5875	8561.9653	1563.0435	496.88
5	1	-20.54873	5394.6415	4605.6485	11196.467	2040.1992	496.88
6	1	-11.25	5380.255	5274.9709	12769.451	2319.9291	496.88
7	1	-4.84325	5370.3425	5736.4305	13669.747	2455.7717	496.88
8	1	5.15675	5354.8705	6227.6683	15034.834	2726.2733	496.88
9	1	12.5	5343.5095	6410.1298	16005.783	2970.3508	496.88
10	1	16.5	5337.321	6509.3953	16484.376	3087.7725	496.88
11	1	18.53	5334.157	6561.2733	17123.372	2830.1058	0
12	1	24.53	5324.6115	6726.5708	18001.409	3021.0837	0
13	1	35	5307.9545	7015.0865	19452.47	3332.5868	0
14	1	42.5	5300	6973.2	27434	5482.4548	0
15	1	48.79565	5300	6521.6761	27171.894	5533.2092	0
16	1	54.29565	5300	6249.3033	26896.764	5532.4705	0
17	1	60.5	5300	6249.2222	26443.333	5410.9958	0
18	1	75.25	5300	6248.7805	25539.024	5168.8053	0
19	1	92.75	5300	6248.6207	24588.276	4914.0958	0
20	1	111.34435	5300	6248.4904	23892.091	4727.5884	0
21	1	133.78305	5300	6247.7804	23342.533	4580.5252	0
22	1	148.9387	5300	6247.5916	22971.567	4481.1757	0
23	1	158.75	5300	6247.4783	22792.174	4433.1378	0
24	1	178.66665	5300	6247.0596	22366.238	4319.1208	0
25	1	207	5300	6246.7066	21672.355	4133.2902	0
26	1	235.33335	5300	6246.0007	20978.473	3947.5541	0
27	1	263.66665	5300	6245.6478	20284.944	3761.818	0
28	1	292	5300	6244.9419	19591.061	3576.0819	0
29	1	320.33335	5300	6244.589	18897.532	3390.3458	0
30	1	348.66665	5300	6243.8831	18203.649	3204.6097	0
31	1	377	5300	6243.5301	17510.12	3018.8736	0
32	1	405.33335	5300	6242.8243	16816.237	2833.1375	0
33	1	426.25	5300	6242.5185	16242.963	2679.611	0
34	1	446.2	5300	6242.0455	15693.182	2532.4244	0
35	1	472.6	5300	6241.6667	15046.97	2359.3738	0
36	1	499	5300	6241.2879	14400.379	2186.2218	0

37	1	525.4	5300	6240.9091	13754.167	2013.1713	0
38	1	551.8	5300	6240.1515	13107.576	1840.1208	0
39	1	582.5	5300	6021.7143	12324.857	1688.922	0
40	1	607.5	5305.8335	5345.4943	13202.182	2105.193	0
41	1	630	5323.3335	4160.4121	10174.223	1611.3958	0
42	1	658.1579	5342.5	2965.1433	7669.3734	3054.9628	0
43	1	684.4737	5357.5	2029.837	5216.7982	2069.6368	0
44	1	710.7895	5372.5	1094.5636	2764.0579	1084.1823	0
45	1	732.76665	5385.027	313.44999	703.89445	148.31593	0

CHATFIELD DAM
Steady-Seepage Stability Re-Analysis

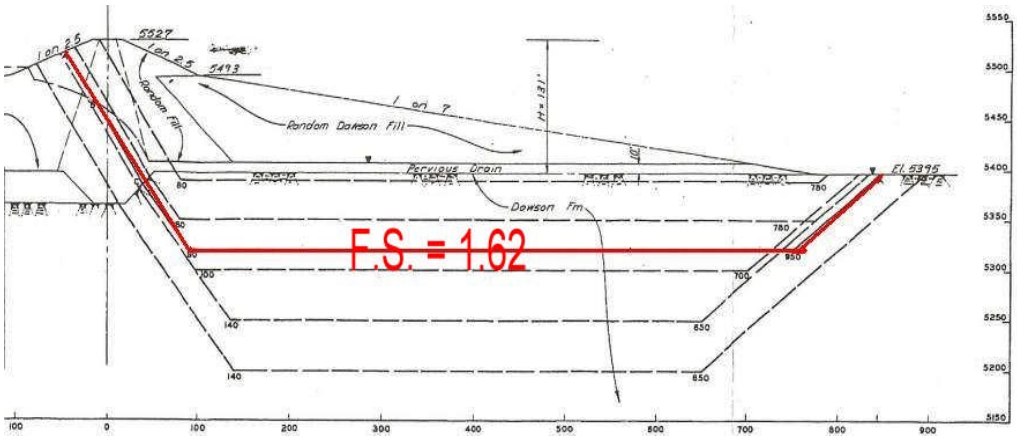

Figure C-2a: Original Stability Analysis from "Embankment Criteria and Performance Report," April, 1980.

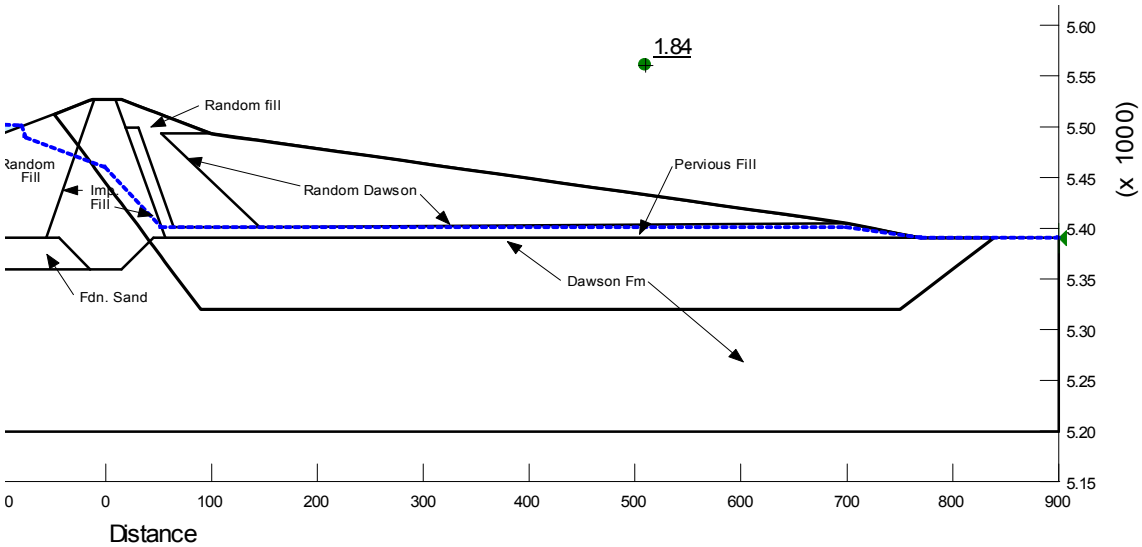

Figure C-2b: Re-Analysis Output from Slope/W

Figure C-2: Steady-Seepage Re-Analysis, Station 68+50

Station 68+50, Steady Seepage, Pool El. 5500

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File Information

Title: Chatfield Dam, Post Liquefaction Stability Assessment.
Comments:
Created By: R.L. Donovan
Revision Number: 50
Last Edited By: R. L. Donovan
Date: 09/08/2009
Time: 10:10:16 AM
File Name: Chatfield Sta 68+50 Steady Seepage.gsz
Directory: F:\Projects\Other\Chatfield Dam\
Last Solved Date: 09/08/2009
Last Solved Time: 10:10:22 AM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Station 68+50 Pool 5500

Description: See Plate B-6 of Draft Water Supply Re-Allocation study.
Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

SlipSurface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Fully-Specified

Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
FOS Distribution
FOS Calculation Option: Constant
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Embankment Impervious

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 24.3 °
Pore Water Pressure
Piezometric Line: 1

Embankment Dawson

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 22.3 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Embankment Random

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 24.3 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Embankment Pervious

Model: Mohr-Coulomb

Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 36.1 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Sand

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Dawson

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 15 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Limits

Left Coordinate: (-800, 5390) ft
Right Coordinate: (900, 5390.0789) ft

Fully Specified Slip Surfaces

Fully Specified Slip Surface 1

	X (ft)	Y (ft)
	-50	5514
	90	5320
	750	5320
	845	5395

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	-800	5499.9853
	-79.5642	5500.1743
	-75.8814	5490.203
	0.3135	5460.2447
	52.5913	5400.1504
	700	5400
	770	5390
	900	5390.0789

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	1	1.84	(407.58, 5543.05)	396.191	(-48.88, 5512.45)	(838.72, 5390.04)

Slices of Slip Surface: 1

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
1	1	-37.280165	5496.374	-1332.1226	1666.7171	752.55163	0
2	1	-20.725875	5473.4345	-306.85143	4045.3081	1826.5266	0
3	1	-14.13571	5464.3025	101.30585	5013.0599	2217.742	0
4	1	-11.25	5460.3035	280.02287	5424.4337	2322.7906	0
5	1	-4.84325	5451.4255	676.84842	6220.9457	2503.2559	0
6	1	5.15675	5437.5685	1067.6088	7416.2321	2866.5133	0
7	1	12.5	5427.393	1175.8714	8257.5499	3197.5005	0
8	1	17.03	5421.116	1242.5819	8711.1899	3372.2058	0
9	1	24.53	5410.723	1353.1476	9437.8529	3650.3844	0
10	1	35.898205	5394.9695	1520.6556	10488.426	4049.1038	0
11	1	43.398205	5384.5765	1631.2195	12009.697	2780.9045	0

12	1	48.79565	5377.0975	1710.7767	12558.904	2906.7468	0
13	1	54.29565	5369.476	1913.9937	13123.106	3003.4726	0
14	1	60.5	5360.8785	2450.4315	13703.093	3015.1414	0
15	1	77.5	5337.3215	3920.2734	15449.36	3089.2095	0
16	1	95	5320	5000.8	22129	4589.4874	0
17	1	111.21795	5320	5000.4747	21683.581	4470.2248	0
18	1	133.6538	5320	5000.029	21280.655	4362.3805	0
19	1	151.9117	5320	4999.9254	20953.82	4274.8333	0
20	1	173.98085	5320	4999.6274	20559.753	4169.3232	0
21	1	204.0391	5320	4999.2947	20023.128	4025.6241	0
22	1	234.0973	5320	4998.6293	19486.504	3882.0142	0
23	1	264.15555	5320	4998.2966	18949.879	3738.3151	0
24	1	294.2138	5320	4997.964	18413.254	3594.6161	0
25	1	324.27205	5320	4997.2986	17876.629	3451.0061	0
26	1	354.3303	5320	4996.9659	17340.004	3307.3071	0
27	1	384.3885	5320	4996.6332	16803.379	3163.608	0
28	1	414.44675	5320	4996.3005	16266.754	3019.9089	0
29	1	444.505	5320	4995.6351	15730.129	2876.299	0
30	1	474.5632	5320	4995.3025	15193.504	2732.6	0
31	1	504.62145	5320	4994.9698	14656.879	2588.9009	0
32	1	534.6797	5320	4994.3044	14120.255	2445.291	0
33	1	564.73795	5320	4993.9717	13583.63	2301.5919	0
34	1	594.7962	5320	4993.639	13047.005	2157.8928	0
35	1	624.8544	5320	4992.9736	12510.38	2014.2829	0
36	1	654.91265	5320	4992.641	11973.755	1870.5839	0
37	1	684.9709	5320	4992.3083	11437.13	1726.8848	0
38	1	712.5	5320	4880.4	10816	1590.4392	0
39	1	737.5	5320	4657.6	10110.4	1461.0734	0
40	1	760	5327.8945	3964.4432	10272.939	1690.3564	0
41	1	787.17985	5349.3525	2536.9533	6496.1171	1060.8548	0
42	1	821.5396	5376.479	845.67393	2165.3419	353.60397	0

APPENDIX D

POST-LIQUEFACTION STABILITY ANALYSES

Table D-1: Adopted Shear Strengths

Material	Cohesion (psf)	Friction angle (degrees)
Embankment		
Impervious	0	24.3
Random	0	24.3
Pervious	0	36.1
Dawson Fm.	0	22.3
Foundation		
Clay	0	20.8
Sand	0	33.0
Dawson Fm.	0	15.0
Liquefied Zones		
Upstream	680	0
Downstream	790	0

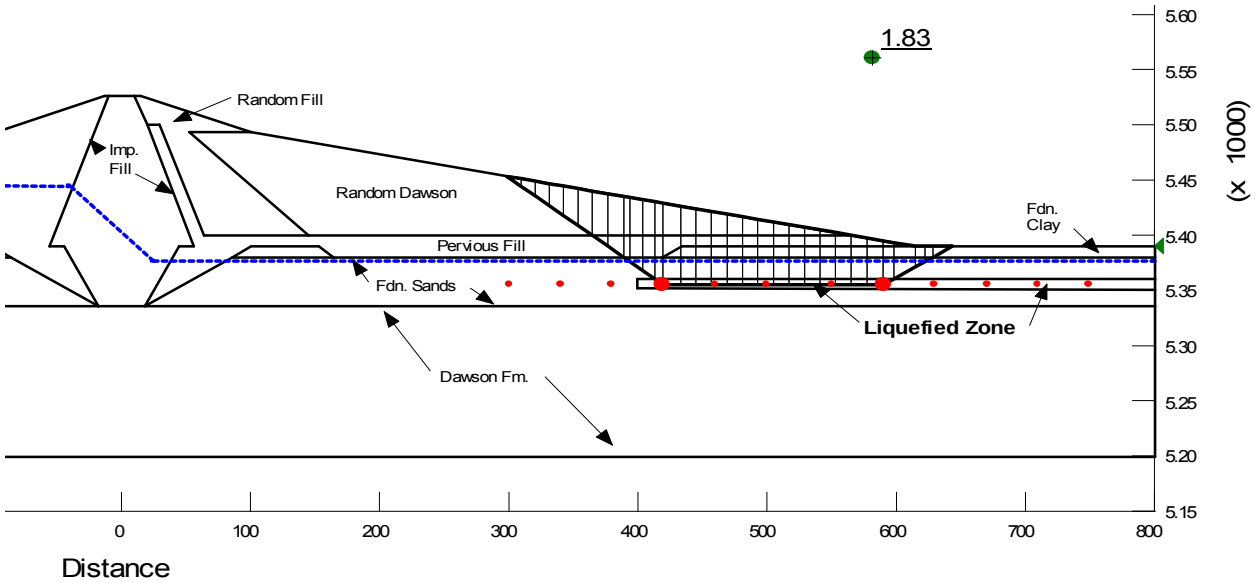

Figure D-1: Post-Liquefaction Slope Stability at Downstream Toe

Figure D-1: Sta 95+00, Pool El. 5444, Downstream Toe

Station 95+00 Pool El. 5444 Downstream Toe

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File Information

Title: Chatfield Dam, Post-Liquefaction Stability Assessment.
Comments:
Created By:
Revision Number: 49
Last Edited By: R.L. Donovan
Date: 09/01/2009
Time: 10:57:48 AM
File Name: Chatfield Sta 95 Pool 5444 - Min Liq - Max S - DS.gsz
Directory: F:\Projects\Other\Chatfield Dam\
Last Solved Date: 09/01/2009
Last Solved Time: 10:57:58 AM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Station 95+00 Pool 5444

Description: See Plate B-73 for referenced data.
Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

SlipSurface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Block

Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
FOS Distribution
FOS Calculation Option: Constant
Restrict Block Crossing: No
Advanced
Number of Slices: 30
Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Embankment Impervious

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 24.3 °
Pore Water Pressure
Piezometric Line: 1

Embankment Dawson

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 22.3 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Embankment Random

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 24.3 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Embankment Pervious

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 36.1 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1

Foundation Clay

Model: Mohr-Coulomb
Unit Weight: 118 pcf
Cohesion: 0 psf
Phi: 20.8 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1

Foundation Sand

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1

Foundation Dawson

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 15 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1

Liquefied U.S. Foundation Sand

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 680 psf
Phi: 0 °
Phi-B: 0 °
Pore Water Pressure
    Piezometric Line: 1

Liquefied D.S. Foundation Sand

Model: Mohr-Coulomb

Unit Weight: 125 pcf
Cohesion: 790 psf
Phi: 0 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Limits

Left Coordinate: (-800, 5390) ft
Right Coordinate: (800, 5390.0789) ft

Slip Surface Block

Left Grid
Upper Left: (300, 5355) ft
Lower Left: (300, 5355) ft
Lower Right: (500, 5355) ft
X Increments: 5
Y Increments: 0
Starting Angle: 135 °
Ending Angle: 147 °
Angle Increments: 2
Right Grid
Upper Left: (550, 5355) ft
Lower Left: (550, 5355) ft
Lower Right: (750, 5355) ft
X Increments: 5
Y Increments: 0
Starting Angle: 33 °
Ending Angle: 45 °
Angle Increments: 3

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	-800	5444
	-40	5444
	25	5376
	800	5376

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	245	1.83	(479.972, 5469.09)	159.547	(298.646, 5453.27)	(643.914, 5390.01)

Slices of Slip Surface: 245

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
1	245	304.1276	5448.8315	-4544.7103	320.35611	131.38762	0
2	245	315.0916	5439.953	-3990.7003	961.0825	394.16866	0
3	245	326.0556	5431.0745	-3436.6904	1601.7805	656.93808	0
4	245	337.0196	5422.196	-2882.6096	2242.5495	919.73656	0
5	245	347.9836	5413.3175	-2328.5996	2883.2475	1182.506	0
6	245	358.9476	5404.439	-1774.5897	3523.9455	1445.2754	0
7	245	370.6041	5395	-1185.5765	3873.4034	2824.5344	0
8	245	382.9531	5385	-561.59913	4604.7365	3357.8316	0
9	245	391.5974	5378	-124.80053	5223.8312	3392.3956	0
10	245	398.8723	5372.109	242.80206	5686.5415	3535.2057	0
11	245	408.4825	5364.327	728.41427	6327.8948	3636.3452	0
12	245	416.3938	5357.9205	1128.1772	7860.5251	0	790
13	245	426.25	5355.2025	1297.7868	8974.4833	0	790
14	245	439	5355	1310.4167	8680.8333	0	790
15	245	451	5355	1310.4167	8393.3333	0	790
16	245	463	5355	1310.4167	8105.1667	0	790
17	245	475	5355	1310.4167	7817.1667	0	790
18	245	487	5355	1310.4167	7529.1667	0	790
19	245	499	5355	1310.4167	7241.1667	0	790
20	245	511	5355	1310.4167	6953.1667	0	790
21	245	523	5355	1310.4167	6665.1667	0	790
22	245	535	5355	1310.4167	6377.1667	0	790
23	245	547	5355	1310.4167	6089.1667	0	790
24	245	559	5355	1310.4167	5801.1667	0	790
25	245	571.25	5355	1310.4	5494.64	0	790
26	245	583.75	5355	1310.4	5169.68	0	790

27	245	594.2483	5357.759	1138.2794	5243.8634	0	790
28	245	606.7483	5365.8765	631.7207	4610.5295	2583.8687	0
29	245	618.6686	5373.6175	148.66438	2990.2047	1845.3178	0
30	245	625.4169	5378	-124.79995	2207.9666	1433.8703	0
31	245	636.20545	5385.006	-561.97347	775.20898	294.47427	0

Table D-2: Adopted Shear Strengths

Material	Cohesion (psf)	Friction angle (degrees)
Embankment		
Impervious	0	24.3
Random	0	24.3
Pervious	0	36.1
Dawson Fm.	0	22.3
Foundation		
Clay	0	20.8
Sand	0	33.0
Dawson Fm.	0	15.0
Liquefied Zones		
Upstream	1120	0
Downstream	1120	0

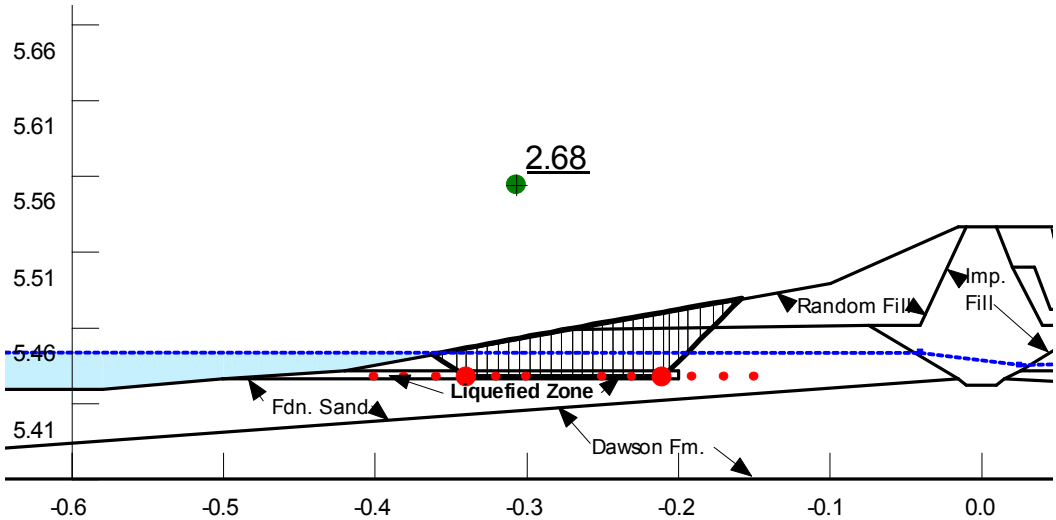

Figure D-1: Post-Liquefaction Slope Stability at Upstream Toe

Figure D-2: Sta 57+60, Pool El. 5444, Upstream Toe

Chatfield Sta 57+60 Pool El 5444 Upstream Toe

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File Information

Revision Number: 21
Date: 09/03/2009
Time: 11:02:31 AM
File Name: Chatfield St 57+60 Pool 5444 - Min Liq - Max S US.gsz
Directory: F:\Projects\Other\Chatfield Dam\
Last Solved Date: 09/03/2009
Last Solved Time: 11:02:42 AM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Chatfield Sta 57+60

Kind: SLOPE/W
Method: Spencer
Settings
- Apply Phreatic Correction: No
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

SlipSurface
- Direction of movement: Right to Left
- Use Passive Mode: No
- Slip Surface Option: Block
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No

FOS Distribution
- FOS Calculation Option: Constant

Restrict Block Crossing: No

Advanced
- Number of Slices: 30
- Optimization Tolerance: 0.01
- Minimum Slip Surface Depth: 0.1 ft
- Optimization Maximum Iterations: 2000
- Optimization Convergence Tolerance: 1e-007
- Starting Optimization Points: 8
- Ending Optimization Points: 16
- Complete Passes per Insertion: 1
- Driving Side Maximum Convex Angle: 5 °
- Resisting Side Maximum Convex Angle: 1 °

Materials

Embankment Impervious

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 24.3 °
Phi-B: 0 °
Pore Water Pressure
- Piezometric Line: 1

Embankment Dawson

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 22.3 °
Phi-B: 0 °
Pore Water Pressure
- Piezometric Line: 1

Embankment Random

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 24.3 °
Phi-B: 0 °
Pore Water Pressure
- Piezometric Line: 1

Embankment Pervious

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 36.1 °

Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Sand

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Foundation Dawson

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 0 psf
Phi: 15 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Liquefied Foundation Sand

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 1120 psf
Phi: 0 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Limits

Left Coordinate: (-800, 5420) ft
Right Coordinate: (800, 5435) ft

Slip Surface Block

Left Grid
Upper Left: (-400, 5428) ft
Lower Left: (-400, 5428) ft
Lower Right: (-300, 5428) ft
X Increments: 5
Y Increments: 0
Starting Angle: 135 °
Ending Angle: 147 °
Angle Increments: 3

Right Grid

Upper Left: (-250, 5428) ft
Lower Left: (-250, 5428) ft
Lower Right: (-150, 5428) ft
X Increments: 5
Y Increments: 0
Starting Angle: 45 °
Ending Angle: 45 °
Angle Increments: 3

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	-800	5444
	-40	5444
	25	5435
	800	5435

Critical Slip Surfaces

	Slip Surface	FOS	Center (ft)	Radius (ft)	Entry (ft)	Exit (ft)
1	372	2.68	(-265.492, 5488.97)	92.704	(-158.286, 5479.71)	(-362.646, 5442.71)

Slices of Slip Surface: 372

	Slip Surface	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
1	372	-359.1799	5440.4555	221.1737	504.06889	183.71428	0
2	372	-350.9369	5435.1025	555.20056	1559.0521	651.90882	0
3	372	-343.07975	5430	873.60099	2615.2204	0	1120
4	372	-336.5	5428	998.4	2507.5714	0	1120
5	372	-329.5	5428	998.4	2670.8571	0	1120
6	372	-322.5	5428	998.4	2834.1429	0	1120

7	372	-315.5	5428	998.4	2997.5714	0	1120
8	372	-308.5	5428	998.4	3160.8571	0	1120
9	372	-301.5	5428	998.4	3324.1429	0	1120
10	372	-294.5	5428	998.4	3487.5714	0	1120
11	372	-287.5	5428	998.4	3650.8571	0	1120
12	372	-280.5	5428	998.4	3814.1429	0	1120
13	372	-273.5	5428	998.4	3977.5714	0	1120
14	372	-266.66665	5428	998.39995	4129.9498	0	1120
15	372	-260	5428	998.39995	4271.3998	0	1120
16	372	-253.33335	5428	998.39995	4412.9998	0	1120
17	372	-246.66665	5428	998.39995	4554.4498	0	1120
18	372	-240	5428	998.39995	4696.0498	0	1120
19	372	-233.33335	5428	998.39995	4837.4998	0	1120
20	372	-226.66665	5428	998.39995	4979.0998	0	1120
21	372	-220	5428	998.39995	5120.5497	0	1120
22	372	-213.33335	5428	998.39995	5262.1497	0	1120
23	372	-208	5430	873.59511	4124.2005	0	1120
24	372	-203	5435	561.59602	3456.456	1879.944	0
25	372	-197	5441	187.19474	2937.3217	1785.9533	0
26	372	-189.76165	5448.2385	-264.47634	2351.7998	1527.2767	0
27	372	-181.28495	5456.715	-793.42067	1699.8892	1103.921	0
28	372	-173.9198	5464.08	-1253.0556	1201.382	542.44479	0
29	372	-167.6662	5470.334	-1643.2645	720.84054	325.47198	0
30	372	-161.41255	5476.5875	-2033.4734	240.27641	108.48896	0