McCall

PARK MEADOWS/CHEYENNE CREEK (SPRING RUN)

DRAINAGE IMPROVEMENTS PRELIMINARY DESIGN

OCTOBER, 1985

S & G NO. 85433

CONSULTING ENGINEERS

SELLARDS & GRIGG, INC.

One Union Square • 143 Union Boulevard, Suite 280

Lakewood, Colorado 80228 (303) 986-1444

SPRING RUN DRAINAGE

BILL — I WILL SIGN AN EASEMENT FOR YOUR PLANNED DRAINAGE ANYTIME — Buz Rieger

Enterprise Development Company

28 UPLAND ROAD

COLORADO SPRINGS, COLORADO, 80906

HOME - 632-0331

N.A. RIEGER

PRESIDENT

WILLOWBROOK APTS —

OFFICE - 473-3711

Spring Run

Easement granted

fall of 1981

by N.A. Rieger

Vern Hammerstoffer project.

I	195,095
II	566,220
III	265,758
IV	273,161
V	6,981

CONSULTING ENGINEERS
SELLARDS & GRIGG, INC.
One Union Square • 143 Union Boulevard, Suite 280
Lakewood, Colorado 80228 (303) 986-1444

October 29, 1985

Mr. William M. McCall, P.E.
City of Colorado Springs
Department of Public Works
City Engineering Division
30 South Nevada
Suite 403
Colorado Springs, CO 80901

Re: Final Report for Park Meadows/Cheyenne Creek (Spring Run)
Drainage Improvements Preliminary Design
S&G No. 85433-26

Dear Mr. McCall:

In accordance with our agreement, 85-112, we have completed the preliminary design for drainage improvements on a portion of the Spring Run channel. Transmitted herewith are seven copies of the final report. The final document provides conceptual drawings and cost estimates for each of the structural flood control alternatives considered and assesses the various impacts of the proposed drainage improvement alternatives. Drainage improvements have been prioritized and construction phasing recommendations have been made relative to identified flooding problems.

We would like to acknowledge the sub-consultant services provided by William Wenk Associates, a landscape architecture consultant, and A. G. Wassenaar, Inc., a geotechnical consultant. The landscape architecture services provided by William Wenk Associates were invaluable in the development of drainage improvement alternatives for Stratton Meadows Park that enhanced the recreational and aesthetic qualities of the Park while accomplishing the flood control objectives. The results of the geotechnical investigation provided by A. G. Wassenaar, Inc., proved to be valuable in assessing technical feasibility of various drainage improvement alternatives. The geotechnical investigation also resulted in design recommendations that will be useful in future final design phases.

Thank you for this opportunity to provide professional services to the City of Colorado Springs. We appreciate the assistance provided by yourself and other members of the City staff.

Very truly yours,
SELLARDS & GRIGG, INC.

Tim G Flanagan

Timothly G. Flanagan, E.I.T.
Project Engineer

Chuck McKnight

Charles A. McKnight, P.E.
Project Manager

CAM:mc
Encl.

PARK MEADOWS/CHEYENNE CREEK

(SPRING RUN)

DRAINAGE IMPROVEMENTS

PRELIMINARY DESIGN

OCTOBER, 1985

Prepared by:

Sellards & Grigg, Inc.
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CHAPTER I

INTRODUCTION

Contract Authorization

The preparation of this preliminary design of drainage improvements for a portion of sub-basin IV (Spring Run), of the Southwest Area Drainage Basin was authorized under the terms of a contract between the City of Colorado Springs and Sellards & Grigg, Inc. dated July 23, 1985.

Project Limits

The upstream project limit is located on the Spring Run channel approximately 700 feet upstream of the Mt. Werner Circle cul-de-sac in an existing concrete lined trapezoidal channel. The downstream project limit is approximately 2,300 feet downstream of the Mt. Werner Circle cul-de-sac on the Spring Run channel.

Nature and Purpose of Study

This preliminary design investigation is intended to provide the City of Colorado Springs with a thorough analysis of the hydraulic capacities of existing drainage facilities within the study area and to develop alternative designs for solving the identified flooding problems. Cost estimates and evaluation of neighborhood impacts, traffic impacts, and utility impacts are analyzed for each of the alternative flood control designs that have been developed for consideration. Landscape architecture considerations have been integrated into the alternative flood control designs in an effort to "soften" the visual impacts of channel improvements.

Scope of Work

The agreed upon Scope of Work for the preliminary design of drainage improvements for a portion of sub-basin IV (Spring Run), of the Southwest Area Drainage Basin is stated as follows for Phase I professional engineering services.

Items of Work

The selected consultant will perform the following items of work:

Phase I -

Item 1 Determine runoff quantities during the 5 year and 100 year storms, from the Southwest Area Drainage Basin Study.

Item 2 Perform hydraulic calculations to determine capacities of existing facilities including the existing concrete channel, and to check for superelevation around sharp bends.

Item 3 Compare existing capacities with 5 year and 100 year flows, and identify flooding problems.

Item 4 Recommend drainage improvements such as channel reconstruction or other items which will reduce or eliminate flooding problems identified in Item 3.

Item 5 Gather any necessary field survey data which is not already available. All survey notes and topographic maps presently at the City Engineer's Office will be made available to the consultant.

Item 6 Provide cost estimates and other pertinent information such as neighborhood impact, traffic impact, and utility conflicts, to be used in evaluating alternatives.

Item 7 Prepare and submit a draft Pre-design Study containing all the above for review by the City Engineer.

Item 8 The draft report shall contain at least two or more designs for consideration.

Item 9 Because of the sensitivity of Meadows Park, and the significant impact the channel will have on the park, the selected consultant will explore the possibility of some "soft" channel lining solutions with less visual impact, in addition to concrete or "hard" linings.

Item 10 Provide geotechnical analysis and recommendations as necessary for preparation of alternatives.

Item 11 Prepare and submit a final Pre-design Study after review and comment of the draft study by the City Engineer. Upon acceptance of the final Pre-design Study, the consultant will provide 6 additional copies to the City.

Previous Studies/Investigations

The only significant basin-wide investigation for Spring Run preceeding this preliminary design investigation is the "Engineering Study of Southwest Area Drainage Basin (Cheyenne Creek, Cheyenne Run, and Spring Run) Colorado Springs, Colorado" prepared by Lincoln DeVore Testing Laboratory, Inc. (Ref. 1). This engineering study was submitted on February 29, 1984 and was approved by the City Council on July 10, 1984.

Existing Drainage Facilities

For the purpose of discussing existing drainage facilities and proposed future drainage improvements, the following reach designations have been established:

Reach 1 Lower Project Limit (Station 19+20) to Station 23+00

Reach 2 Station 23+00 to the downstream end of the City Park (Station 34+80)

Reach 3 Station 34+80 to the downstream end of the upper Mt. Werner Circle crossing of Spring Run (Station 41+92)

Reach 4 Station 41+92 to the downstream end of the trapezoidal concrete channel (Station 48+10)

Reach 5 Station 48+10 to the upstream project limit (Station 50+00)

All stationing references are made from the preliminary design plan and profile drawings that are included in this report as Drawings 2 through 16 in Appendix A.

The existing drainage facilities are summarized in Table I-1 for each of the defined reaches.

TABLE I-1

Existing Drainage Facilities

Reach	From (STA)	To (STA)	Existing Channel Characteristics: Cross Section	Lining	Longitudinal Slope (%)	Side Slope	Bottom Width (feet)	Contributing Point Discharges: Location (STA)	Structure
1	19+20	23+00	Trapezoidal	Concrete	0.61	1:1	8	20+35	Trapezoidal Concrete Channel
2	23+00	24+38	Trapezoidal	Concrete	0.94	1:1	Varies 6 to 19	24+00	Concrete Conduit
2	24+38	24+88	Twin Cell Box Culvert 2-(9' x 3.5')	Concrete	0.40	Vertical	19	-	-
2	24+88	34+80	Non-Prismatic	Natural Vegetation	1.05	Varies	Varies	-	-
3	34+80	41+92	Non-Prismatic	Grass	0.82	Varies	Varies	-	-
4	41+92	42+41	Twin Cell Box Culvert 2-(11' x 4')	Concrete	0.20	Vertical	23	-	-
4	42+41	43+50	Trapezoidal	Concrete	4.42	Varies	Varies 9 to 23	-	-
4	43+50	46+90	Non-Prismatic	Grass	1.71	Varies	Varies	-	-
4	46+90	48+10	Non-Prismatic	Concrete	0.68	Varies (1:1 to Vertical)	8	-	-
5	48+10	50+00	Trapezoidal	Concrete	0.57	1:1	8	-	-

CHAPTER II

DETERMINATION OF DESIGN DISCHARGE FOR DRAINAGE IMPROVEMENTS

The "Engineering Study of Southwest Area Drainage Basin (Cheyenne Creek, Cheyenne Run, and Spring Run) Colorado Springs, Colorado" (Ref. 1) prepared by Lincoln DeVore Testing Laboratory, Inc. was used to obtain the 5-year and 100-year peak discharges for the project area. This study provided a comprehensive analysis of the Spring Run watershed. The 5-year and 100-year design discharges throughout the study area were reported to be 138 cfs and 465 cfs respectively. The City of Colorado Springs criterion normally requires that the 5-year discharge be used for the design of drainage improvements and appurtenances unless the 100-year discharge exceeds 500 cfs. The design discharge using this criterion would have been 138 cfs. Sellards & Grigg, Inc., however, was advised by the City of Colorado Springs in a letter dated August 5, 1985 to adopt the 100-year discharge of 465 cfs as the design discharge.

CHAPTER III

IDENTIFICATION OF EXISTING DRAINAGE PROBLEMS

The existing channel throughout the project area was analyzed using the HEC-2 computer model developed by the United States Army Corps of Engineers (Ref. 2). Field surveyed cross sections of the channel and overbank areas were obtained to provide the necessary geometric data to use in the HEC-2 model. Throughout most of the project area, the channel is "perched" with no positive drainage toward the channel in the transverse direction on one or both sides of the channel. The overflows from the "perched" reaches of the channel may, therefore, become separated from the flow in the main channel and enter the main channel again at some point downstream. The channel discharge capacity for the "perched" channel reaches was determined by analyzing multiple discharges with the HEC-2 model to determine the "incipient overflow" discharge at critical locations throughout the project area. For the reaches of the channel that were not "perched" allowing for positive drainage toward the channel in the transverse direction, the water surface profile for the 5-year and 100-year peak discharges was determined taking full account of overbank flow. The only reach of existing channel that exhibited positive transverse drainage toward both sides of the channel was in the park from Station 37+00 to Station 41+92. The 100-year flood plain in this reach of the park does not result in the inundation of any inhabitable structures. Table III-1 provides a basic summary of the critical locations of overflow and the "incipient overflow discharge" for the "perched" reaches of the channel as well as a summary of average flow depth and top width for the channel reach that is not "perched".

From Table III-1 it can be observed that all reaches of the existing channel except Reach 5 do not, at some point, have sufficient capacity to pass the 100-year discharge of 465 cfs. Reach 5 has sufficient capacity to pass the 100-year discharge of 465 cfs with essentially no freeboard. Additional freeboard would be provided on the north side of the channel by construction of a berm. The sharp bend in Reach 5 from Station 49+00 to Station 50+00 results in some superelevation of the flow on the north side of the channel. The superelevation of flow was analyzed by application of Newton's second law of motion to the centrifugal action in the curve as presented in the Open Channel Hydraulics textbook by Chow (Ref. 3). The maximum superelevation predicted for the sharp bend in Reach 5 from Station 49+00 to Station 50+00 with a 100-year discharge of 465 cfs was 0.41 feet. "Splash walls" 1 foot high have been constructed in the bend from approximately Station 49+00 to Station 50+00 thus eliminating any overflow through the bend.

The two box culverts in the project area that cross the Spring Run channel on Mt. Werner Circle were found to have adequate capacity to pass the 100-year peak discharge with an unobstructed waterway. The provision of an unobstructed waterway will require relocation of gas lines that go through both cells of both box culverts.

The channel capacity inadequacies determined by mathematical analysis were further verified by field reconnaissance efforts following the severe thunderstorm of July 19, 1985. High water marks on fences and trees could be observed near the Pebble Creek apartments which are near the downstream end of the park (Station 34+80) and just upstream of the channel transition at Station 47+20. The peak discharge resulting from the thunderstorm of July 19, 1985 is unknown, however, it is noteworthy that high water marks were observed on fences near Station 48+10 at the downstream end of Reach 5. Reach 5 was determined to have adequate capacity to pass the 100-year discharge of 465 cfs by mathematical analysis; the observed high water marks are believed to be the result of "backwater" created by the inadequate channel transition from trapezoidal channel to rectangular channel from Station 47+30 to Station 47+55.

Overflow high water marks could also be observed outside of the channel in the immediate upstream vicinity of the upper Mt. Werner box culvert indicating an overflow across Mt. Werner Circle. The overflow was, to a large degree, the result of severe debris blockage in the box culvert caused by the existing gas line running through the box culvert in a transverse direction.

TABLE III-1

Summary of Existing Channel Overflow
Locations, Discharges,
Flood Plain Depths, and Top Widths

Reach	Location (STA)	"Perched" Condition: Minimum Incipient Overflow Discharge (cfs)	"Non-Perched" Condition: Avg. Flood Plain Depth (ft)	"Non-Perched" Condition: Avg. Flood Plain Topwidth (ft)
1	20+00	190	-	-
2	29+00	50	-	-
3	35+00 to 36+00	20	-	-
3	38+00	-	3.2	95
4	45+00	225	-	-
5	48+10 to 50+00	465+	-	-

CHAPTER IV

HYDRAULIC DESIGN CRITERIA

The City of Colorado Springs has no formal design criteria for channel improvements. Sellards & Grigg, Inc. developed the following hydraulic design criteria based primarily on past experience and the existing hydraulic design criteria of the Urban Drainage and Flood Control District (Ref. 4) which serves the Denver Metropolitan area. The hydraulic design criteria summarized in Table IV-1 were presented to the City of Colorado Springs and adopted for use on this project during a meeting held on August 30, 1985. The hydraulic design criteria is intended to apply to the design of channel improvements rather than to the evaluation of adequacy for existing channels.

TABLE IV-1

Hydraulic Design Criteria

	Grass-lined Channel	Riprap-lined Channel	Concrete-lined Channel	Underground Conduits
Mannings "n" Roughness Coefficient	.030-.035	.045	.015	.013
Maximum Average Velocity	7 fps	12 fps	-	-
Maximum Permissible Velocity	7.5 fps	-	15 fps	-
Minimum Permissible Velocity	2.0 fps	-	2.0 fps	2.0 fps
Side Slopes	$\leq$3:1	$\leq$2:1	2:1 to Vertical	-
Freeboard	1' min.	1' min.	1' min.	HGL below Ground Surface
Froude Number	$\leq$0.8	$\leq$0.8	$\leq$0.8 (for subcrit. flow only)	-
Low Flow Channel Capacity	1% to 3% of Q(100)	1% to 3% of Q(100)	1% to 3% of Q(100)	-
Vertical Drop Structure Design	Max. Height=4 ft.	Max. Height=4 ft.	Max. Height=4 ft.	-

CHAPTER V

GEOTECHNICAL EVALUATION

A geotechnical investigation of the project area was performed in accordance with Item 10 of the Scope of Work. The geotechnical investigation was performed by A. G. Wassenaar, Inc. as a sub-consultant to Sellards & Grigg, Inc. The drainage improvement alternatives developed by Sellards & Grigg, Inc. are compatible with the recommendations of the geotechnical investigation by A. G. Wassenaar, Inc. The geotechnical report makes several recommendations that should be used for the final design of the proposed drainage improvements. It is noteworthy that the geotechnical investigation has resulted in the conclusion that the surface water in Spring Run is perched in a clay and sand strata near the surface and is not continuous with the underlying ground water table. For this reason, a layer of drainage filter material will probably not be required under the full length of the drainage improvements. Filter material for stabilization has, however, been included in the construction cost estimates in this report to address the contingency that unstable soil conditions will be encountered at various locations beneath the proposed drainage improvements. The geotechnical report in its entirety has been included with this report as Appendix B.

CHAPTER VI

DRAINAGE IMPROVEMENT ALTERNATIVES

Drainage improvement alternatives have been developed for each of the reaches identified in Chapter I. The drainage improvement alternatives that have been developed consist entirely of channel improvements. The channel improvement alternatives have been developed with consideration given to construction feasibility within the limited existing rights-of-way. Particular attention has been given to channel improvement aesthetics and integration of the flood control function with recreational objectives in Meadows Park (Reach 3). Recreational, maintenance, and landscape architecture considerations have been studied in detail by William Wenk Associates, a landscape architecture firm, as a sub-consultant to Sellards & Grigg, Inc.

Table VI-1 gives the general characteristics of the channel improvement alternatives that have been developed for each reach in the project area.

Commentary on Channel Improvements for Reach 1

Station 19+20 to Station 23+00

The right-of-way available in Reach 1 is generally 28 feet wide. The existing channel in Reach 1 has a transverse gradient away from the channel on the east side of the channel and toward the channel on the west side of the channel. The existing channel in Reach 1 is a concrete trapezoidal channel with a bottom width of 8 feet and side slopes of 1 to 1. The existing channel in Reach 1 has a longitudinal invert slope of approximately 0.61 percent.

Alternative 1-A

Alternative 1-A is a 16-foot wide rectangular concrete channel for the full length of Reach 1. In order to allow for design of the channel without a significant backwater effect, it was assumed in the hydraulic calculations that the 16-foot wide channel would ultimately be continued to Montrose Avenue (about 100 feet downstream of Station 19+20), and would not transition to the existing 8-foot wide rectangular concrete channel. The longitudinal invert slope would be 0.2% resulting in a flow depth ranging from 4.2 feet at Station 19+20 to 3.9 feet at Station 23+00 for the design discharge of 465 cfs. The mild longitudinal gradient of 0.2% allows for maintenance of subcritical flow with a maximum Froude Number of 0.8. The channel velocity for the design discharge averages about 7.1 feet per second. A secondary reason for the longitudinal slope in Reach 1 is to allow a greater depth to the invert of the proposed drainage alternatives in Reach 2. There are two bends in the channel with radii of approximately 115 feet and 170 feet at the channel centerline. The predicted superelevation for these bends is 0.11 feet and 0.08 feet respectively.

A rectangular concrete side channel six feet in width brings drainage from a cul-de-sac into the channel at approximately Station 20+35. The gradient of the side channel is mild resulting in flooding of the cul-de-sac during major flood events in the Spring Run channel. Alternative 1-A would have an invert elevation approximately one foot lower than the existing channel invert at the point of confluence with the six foot wide rectangular concrete side channel and the depth of flow for the design discharge of 465 cfs would be less than the depth of flow in the existing channel for the same discharge. Thus, the flooding impact on the cul-de-sac would be reduced by Alternative 1-A relative to the existing channel conditions. Some minor flooding of the cul-de-sac however, would still occur for the design flood discharge with Alternative 1-A.

Commentary on Channel Improvements for Reach 2

Station 23+00 to Station 34+80

The right-of-way available in Reach 2 is generally 28 feet wide. The existing channel in Reach 2 generally has a transverse gradient away from the channel on the north side of the channel and toward the channel on the south side of the channel. The existing channel in Reach 2 is a concrete trapezoidal channel with a bottom width varying from 6 to 19 feet and side slopes varying from 1:1 to vertical from Station 23+00 to Station 24+38. The longitudinal invert slope from Station 23+00 to Station 24+38 is approximately 0.94 percent. The lower box culvert through Mount Werner Circle extends from Station 24+38 to Station 24+88 in Reach 2. Upstream of the box culvert, the existing channel is unlined with an average thalweg slope of approximately 1.05 percent.

Alternative 2-A

Alternative 2-A is a 16-foot wide rectangular concrete channel for the full length of Reach 2. The longitudinal invert slope would be 0.23% resulting in a normal flow depth of 3.5 feet for the design discharge of 465 cfs. As in Reach 1, the mild longitudinal gradient of 0.23% allows for maintenance of subcritical flow with a maximum Froude Number of 0.8. The channel velocity for the design discharge is 8.4 feet per second. The only bend in the channel resulting in significant superelevation is located between Station 23+20 and Station 23+50. The bend radius is 50 feet and the predicted superelevation is 0.27 feet. The invert of the rectangular channel at the upstream end of Reach 2 allows for easy transition to the improved grass lined channel in Reach 3. The rectangular channel alternative would allow for access by pedestrians and bicyclists. Pedestrian and bicycle traffic in the channel could be discouraged by signage and the elimination of any trails leading directly to points of access, however, the need to provide egress from channel at intermediate points in Reach 2 is recognized. The provision of adequate egress from the channel would probably be accomplished by installing side wall ladders at regular intervals throughout Reach 2. The ladders would be designed to provide minimum impedance to flood flows.

Alternative 2-B

Alternative 2-B is 2-60 inch diameter reinforced concrete pipes for the full length of Reach 2. The longitudinal invert slope would vary from 0.20% to 0.81% resulting in a hydraulic grade line above the crown of the pipes for the design discharge of 465 cfs. The velocity in the pipes flowing full would be approximately 12 feet per second. The horizontal alignment would be similar to the horizontal alignment of Alternative 2-A, however, the problem of superelevation at bends would be eliminated because the total design discharge of 465 cfs would be carried inside of the pipes. The alignment of the pipes for Alternative 2-A is shown to be a curvilinear alignment rather than a series of straight sections with sharp bends. The curvilinear alignment would be constructed by installing pipe sections with small joint deflections. The Spring Run base flow which is estimated to be 5 to 10 cfs would be routed into one of the pipes so as to increase the low flow velocities. The routing of low flows into a single pipe could be accomplished by means of a low level weir at the point where the low flow channel in Reach 3 meets the transition structure at the upstream end of Reach 2. The hydraulic grade line would be essentially at the average ground surface throughout Reach 2 for the design discharge of 465 cfs. The safety hazard and potential for debris blockage at the entrance to the pipes at the upstream end of Reach 2 would be dealt with by a trash rack designed to deflect debris in an upward direction so as not to block the entrance to the pipes. The trash rack would also greatly reduce the risk to the individual of being carried into one of the pipes by the localized high velocities that would be present immediately upstream of the pipe entrance during a major flood event.

Alternative 2-C

Alternative 2-C is a single 8 foot wide by 5 foot high box culvert for the full length of Reach 2. The box culvert would in all likelihood be precast and installed in sections. Alternative 2-C would have hydraulic characteristics very similar to Alternative 2-B. The required capacity of 465 cfs, however, would be provided by a single conduit instead of two pipes affording a lesser construction trench width than required by Alternative 2-B. The hydraulic grade line would again be above the crown of the pipes for the design discharge of 465 cfs. However, because of the reduced wetted perimeter of a single conduit, the hydraulic grade line for Alternative 2-C would have a flatter slope than the hydraulic grade line for Alternative 2-B, and would remain below the ground surface throughout Reach 2. Alternative 2-C would also produce a lower water surface in Reach 3 than would Alternative 2-B. The velocity in the reinforced concrete box would be approximately 12 feet per second. As with Alternative 2-B, the problem of superelevation at bends would be eliminated because the total design discharge of 465 cfs would be carried inside of the conduit. The alignment of the conduit for Alternative 2-A is again shown to be a curvilinear alignment constructed as described for Alternative 2-B. The safety hazard and potential for debris blockage at the entrance to the conduit at the upstream end of Reach 2 would be dealt with as discussed for Alternative 2-B. = trash rack

Commentary on Channel Improvements for Reach 3

Station 34+80 to Station 41+92

There is no defined drainage easement for Reach 3, however, since the Stratton Meadows Park is under City ownership, it is assumed that the entire park is available for construction of channel improvements. There are transvere gradients away from the existing Spring Run channel in both directions from Station 37+00 to Station 38+00 in Reach 3. The alternative that has been developed for Reach 3 involves incising the improved channel to a depth of 4 to 5 feet below the existing channel flow from Station 34+80 to Station 37+70. Reach 3 from Station 37+70 to Station 41+92 generally has sufficient transverse gradient toward the existing channel in both directions to allow for maintenance of the existing thalweg slope. The top width of the flood plain for the design discharge of 465 cfs would be contained entirely within the Stratton Meadows Park from Station 37+70 to Station 41+92 without inundation of any inhabitable structure, thus allowing for maintenance of the "natural" cross section in this reach. Minor cross section variations from Station 37+70 to Station 41+92 may be desirable from the standpoint of maintenance, recreational use, and aesthetics. The existing low flow channel in Reach 3 has near vertical side slopes constructed of railroad ties with a "natural" bottom. The railroad ties are deteriorating in many locations throughout Reach 3. The "natural" bottom of the existing low flow channel would contribute to the siltation of the "hard" lining channel improvement alternatives in Reaches 1 and 2. Thus, it is proposed that the entire length of low flow channel be replaced with a "hard" surface concrete low flow channel. The use of concrete for the low flow channel could be made unobtusive to the park by designing the bottom width of the low flow channel such that the base flow in Spring Run (5 to 10 cfs) would cover the concrete surface at the bottom of the low flow channel. The plan to incise the low flow channel in Reach 3 from Station 34+80 to Station 37+70 and construct a low flow channel at essentially the existing grade from Station 37+70 to Station 41+92 requires that a drop structure be constructed at Station 37+70. The drop structure would be integrated with the landscaping of the park and provide the required energy dissipation.

Alternatives 3-A and 3-B have been developed for the Stratton Meadows Park. The two alternatives are similar in scope, with Alternative 3-B being the less costly. The major cost savings for Alternative 3-B result from a slightly reduced scope of improvements and the use of asphalt rather than concrete for the maintenance path. The cost estimate tables in Chapter VII outline the differences between the two alternatives.

It should be noted that the reduced hydraulic efficiency of two conduits (Alternative 2-B) compared to a single conduit (Alternative 2-C) for Reach 2 results in a higher 100-year water surface elevation in the park downstream of Station 37+70. If Alternative 2-B is selected for Reach 2, some revision of the proposed grading for the park will be necessary to accommodate the higher water surface.

<u>Commentary on Channel Improvements for Reach 4</u>

Station 41+92 to Station 48+10

There is no drainage easement of record upstream of Station 43+50 in Reach 4 and this portion of Reach 4 is presently under private ownership. Therefore, a permanent easement would have to be acquired before channel improvements could be constructed in Reach 4. The existing channel in Reach 4 generally has a transverse gradient away from the channel on the north side of the channel and toward the channel on the south side of the channel. Reach 4 includes the upper crossing of Mt. Werner Circle which is a twin cell 11 feet wide by 4 feet high box culvert from Station 41+92 to

Station 42+41. The existing channel in Reach 4 consists of a badly deteriorated trapezoidal concrete channel from Station 42+41 to Station 43+50. The longitudinal invert slope from Station 42+41 to Station 43+50 is approximately 4.42 percent. Upstream of Station 43+50 to Station 46+90 Reach 4 consists of a natural grass-lined channel with an average thalweg slope of 1.71 percent. Reach 4 from Station 46+90 to Station 48+10 consists of a transition from trapezoidal concrete channel to rectangular concrete channel.

Alternative 4-A

Alternative 4-A is a 16-foot wide rectangular concrete channel for the full length of Reach 4. The longitudinal invert slope would be 0.23% for the design discharge of 465 cfs. Due to the extremely steep gradient of the existing channel, it was necessary to provide for drop structures at Stations 43+30, 44+00, and 46+90. The drop structures are vertical walled drop structures approximately 3 feet high with the necessary downstream energy dissipation. Flow in the channel would be subcritical with a maximum Froude number of 0.8. There is a transition at the upstream end of Reach 4 from the rectangular concrete channel proposed for Alternative 4-A to the existing trapezoidal concrete channel of Reach 5. Supercritical flow from Reach 5 would continue to the drop structure at Station 46+90. Flow would be subcritical downstream of Station 46+90. The discouragement of pedestrian and bicycle traffic in the bottom of the channel and the provision for emergency egress from the channel would be dealt with as discussed in the commentary for Alternative 2-A. The normal flow depth for the design discharge is 3.5 feet with a velocity of 8.4 feet per second. There are no significant bends in the channel improvements proposed by Alternative 4-A, thus there is no superelevation to consider.

Alternative 4-B

Alternative 4-B is a trapezoidal concrete channel with an 8 foot bottom width and 1.5:1 side slopes. The longitudinal invert slope is 0.20%. The normal flow depth would be 4.1 feet with a channel velocity of 8.2 feet per second for the design discharge of 465 cfs. Drop structures approximately 3 feet high would be provided at Station 43+50, 44+10, and 46+80, with the necessary downstream energy dissipators. The channel for Alternative 4-B would be rectangular downstream of the drop at Station 43+50 to provide a simple transition to the box culvert at the downstream end of Reach 4. The channel upstream of the drop at Station 46+80 would be a trapezoidal concrete channel with an 8 foot bottom width and 1:1 side slopes. The proposed channel would have a cross section and longitudinal slope (0.70%) essentially matching the existing cross section and slope of Reach 5. This would prevent backwater from affecting the supercritical flow in Reach 5. Downstream of Station 46+80 the flow would be subcritical. The 1.5:1 side slopes allow for egress from the channel without ladders or other special structures.

Commentary on Channel Improvements for Reach 5

Station 48+10 to Station 50+00

The right-of-way available in Reach 5 is uniformly 30 feet wide throughout the full length of the Reach. The existing channel in Reach 5 generally has a transverse gradient away from the channel on the north and east side of the channel and toward the channel on the south and west side of the channel. The existing channel throughout the full length of Reach 5 is a concrete trapezoidal channel with a bottom width of 8 feet and side slopes of 1:1. The longitudinal invert slope is approximately 0.57% percent for the full length of Reach 5. The existing concrete trapezoidal channel in Reach 5 has a capacity essentially equal to the design discharge of 465 cfs. The design flow in the existing channel would be supercritical. It was decided by the City of Colorado Springs to allow exceedance of the Froude Number criteria and retain the existing channel in Reach 5 as an element of the overall improvements plan since the existing channel has a capacity exceeding the design discharge. The existing channel has a 90 degree bend with an approximate radius of 90 feet from Station 48+95 to Station 49+90. Splash walls appoximately one foot high have been constructed from Station 48+95 to Station 49+90. The predicted superelevation in the bend is 0.41 feet. In addition to the superelevation in the bend, circular curves have been found to propogate cross waves in the supercritical flow regime. The height of cross waves that might develop under these circumstances is not easily quantified by mathematical analysis. It is therefore proposed that a berm be constructed on the north and east sides of the channel as an additional factor of safety in the elimination of overflow from the channel near the bend for the design discharge. The construction of such a berm would not interrupt the flow of local drainage laterally toward the channel since the gradient is away from the channel on this side of the channel.

TABLE VI-1

Channel Improvement Alternatives

Reach-Alternative	Type of Conveyance	Lining	Conduit Dimensions (ft)	Channel Bottom Width (ft)	Channel Side Slope	Longitudinal Slope (%)	Normal Depth of Flow for Q=465 cfs (ft)
1-A	Channel	Concrete		16	Vertical	0.2	3.6*
2-A	Channel	Concrete		16	Vertical	0.23	3.5
2-B	Conduit	Concrete	2-5 foot Dia. RCP's	-	-	Varies 0.21 to 0.81	5 feet**
2-C	Conduit	Concrete	8 foot x 5 foot RCB	-	-	Varies 0.21 to 0.81	5 feet**
3-A	Channel	Grass	-	Varies	Varies	0.8	
4-A	Channel	Concrete	-	16	Vertical	0.23	3.5
4-B	Channel	Concrete	-	8	1.5:1	0.20	4.1

*Flow Depth is not normal due to backwater caused by sidewalk on Montrose Avenue.

**The Hydraulic Grade Line (HGL) is above the inside top of the conduit

NOTE: The channel improvement alternatives for Reach 5 consists of berm construction outside of the existing channel construction.

CHAPTER VII

EVALUATION OF DRAINAGE IMPROVEMENT ALTERNATIVES

The drainage improvement alternatives that have been identified are all viable alternatives in terms of their technical feasibility. The selection of alternatives will be based on the cost of the alternatives, which provides for a quantitative comparison of alternatives, as well as the less quantifiable traffic and neighborhood impacts. The conventional criterion which is used to evaluate the feasibility of construction of drainage improvements is the "benefit/cost" ratio. For the purposes of this master planning and preliminary design analysis, the flood control benefits can be assumed to be equal for all alternatives considered since all alternatives offer 100-year flood protection. Traffic, neighborhood and utility impacts have been summarized qualitatively in the paragraphs that follow. Generalized cost estimates for each of the drainage improvement alternatives are provided in Table VII-1 through VII-5.

Estimates of project costs have been prepared by estimating construction quantities and researching unit costs for all construction items that could be readily identified in this preliminary design phase. An engineering and construction management cost equal to 15% of the estimated construction cost and a contingency cost equal to 10% of the construction cost have been added to each cost estimate.

Traffic Impacts

The alternatives presented in Chapter VI would all have some traffic impact on the neighborhoods in the vicinity of the project regardless of the reach or reaches where construction was taking place. The traffic impacts that would result from construction of the proposed channel improvement alternatives can generally be classified as being either minimal or disruptive depending on the reach where construction is taking place and the alternative selected. A minimal traffic impact would be defined as a traffic impact resulting from increased traffic volume and noise level. The minimal traffic impact would not involve any significant detouring or disruption to the flow of traffic. Minimal traffic impacts would exist for the duration of construction for the following reach alternatives.

Alternative 1-A
Alternative 2-A
Alternative 3-A
Alternative 4-A
Alternative 4-B
Alternative 5-A

Minimal traffic impacts

A disruptive traffic impact would be defined as a traffic impact that results in significant detouring of traffic and hindrance to the flow of traffic. Disruptive traffic impacts would exist for the duration of construction for the following reach alternatives.

Alternative 2-B
Alternative 2-C

Disruptive traffic impacts

The construction of the alternatives that have been identified for Reach 2 would involve disruptive traffic impacts due primarily to the construction of new cross drainage through the lower crossing of Spring Run by Mt. Werner Circle. Mt. Werner Circle at this location could either be closed for the duration of construction or restricted to one lane at the option of the City.

Neighborhood Impacts

The alternatives presented in Chapter VI would all have some direct neighborhood impacts aside from the traffic impacts that have been identified. The neighborhood impacts that would result from construction of any of the proposed channel improvement alternatives result primarily from the very restrictive rights-of-way that are available in Reaches 1 and 2. There is presently no available right-of-way for much of Reach 4 and the right-of-way available for acquisition represents only the minimum requirement for construction without impacting existing buildings. Construction activities in Reaches 1, 2, and 4 would in all likelihood involve the use of large pieces of equipment with high noise levels. The most severe neighborhood impact would probably occur in Reach 2 regardless of the alternative selected due to the fact that the 28 foot drainage easement has been encroached upon by the backyard fences of all of the property owners in Reach 2. All of the channel improvement alternatives identified for Reach 2 would require temporary or permanent relocation of the fences to the limits of the easement.

Flood control, which is the primary emphasis of the project, is undoubtedly the greatest single beneficial impact on the adjacent neighborhoods. All of the flood control alternatives would provide 100-year flood protection. Presently adjacent residents experience flooding almost annually.

The beneficial impacts of the channel improvements in the Stratton Meadows Park (Reach 3) are also noteworthy. The channel improvement alternative that has been identified for Reach 3 would significantly enhance the Park as a recreational amenity. The trail system in the park has been conceptually designed to provide increased accessibility to the park while meeting the maintenance requirements of the channel.

As part of the preliminary design work, the study area was assessed for its recreation potential and for potential impacts of channel redevelopment on adjacent uses. Of special interest were potential impacts on Meadows Park. Preliminary inventory and analysis included meeting with representatives of the Parks and Utilities Departments and the community center adjacent to the park to develop goals for the park.

Based on meetings with city staff and field inspections, the following conclusions were drawn.

1. Development of a trail through the study area connecting with the City wide trail system is not consistent with the City trails master plan currently being developed. However, a neighborhood trail would allow residents of the area off-street access to Meadows Park. Prior to the flood in the summer of 1985, a wooden bridge provided easy access from the apartment complex upstream of the Mt. Werner Circle cul-de-sac to Meadows Park. Replacement of the bridge should be considered as part of the channel improvements.

2. Channel construction will have a significant impact on Meadows Park and areas downstream. Impacts on the park include loss of use during construction, and because of change in land forms, potential loss of usable recreation open space in the park. Channel improvements in the residential areas downstream will cause significant loss of vegetation in the existing channel easement but long term use of the area should improve because the channel will be placed in underground concrete conduits. Because conduits have been chosen as the preferred design option, additional permanent channel easements will not be required. Because no property acquisition is required, no long-term disruption of use of individual residences will occur, except for removal of the vegetative screen that now exists between homes along the channel.

3. As part of the master planning process, three Meadows Park design alternatives were developed and reviewed by the City staff. The first alternative proposed minimal improvements through the park. The channel proposed would include a grass-lined trapezoid to accommodate the 100-year flood, low flow channel improvements, and a trail connection from Mt. Werner Circle to the Mt. Werner Circle cul-de-sac. The channel improvements will encroach on the softball field in this alternative, but the field will not be relocated. The drop structure required at Station 37+70 would be a simple concrete structure, designed as required to meet hydraulic requirements.

   The second alternative provides for construction of a trapezoidal channel from the cul-de-sac to Station 37+70, with the widening of the channel and regrading of the park area between Station 37+70 and the end of the park to blend with the existing contours and allow better recreation use. As part of the regrading, earth mounds would be placed on either side of the channel to help contain flood waters exceeding the 100-year flood. As in the first alternative, the drop structure proposed would be simple, serving only engineering uses.

   The third alternative calls for significant regrading of the area downstream of the drop structure, relocation of the existing softball field to maintain full use of the field, and development of a drop structure that makes a maximum use of water in the low flow channel as a recreational resource. Two alternative drop structures were proposed, one "natural" in appearance and built with machine placed boulders, the second a geometric concrete structure of two levels. For maintenance and appearance reasons the concrete structure was chosen as the preferred alternative. The alternative proposes extension of the existing trail system in the park to connect with the

Mt. Werner Circle cul-de-sac upstream, and extending the trail upstream to the channel pedestrian crossing serving the apartment area to the southwest.

Based on the design alternatives provided for the park, City staff recommended that the third development alternative be selected for ultimate channel development improvements. Cost estimates for two variations of the third development alternative are presented as Alternatives 3-A and 3-B in this report.

Multiple use design considerations should be limited to the Meadows Park and the area immediately upstream of the Mt. Werner Circle cul-de-sac. Improvements in the area upstream of the cul-de-sac should address the visual impact of the channel improvements, the safety of children in the area as it relates to the channel, and the provision of a pedestrian crossing to the apartment complex south and west of the channel.

Utility Impacts

The utility impacts are summarized for each of the reach-alternatives as follows:

Reach-Alternative 1-A

There are no significant utility impacts for Reach-Alternative 1-A.

Reach-Alternative 2-A, B, C

Mountain Bell telephone closure risers and cable and underground television cable parallel the channel on both sides. The Mountain Bell telephone cable also crosses the channel at one point. The Mountain Bell telephone cable and the television cable will have to be relocated closer to the boundary of the existing drainage and utility easement. The gas line in the lower Mt. Werner Circle Crossing will require relocation for all Reach-Alternatives, and the water line that parallels the gas line will require relocation for Reach-Alternatives 2-B and 2-C.

Reach-Alternative 3-A

Mountain Bell underground cable continues westward through the park from Reach 2 and will likely require relocation in the area where the channel has been lowered in the park.

Reach-Alternative 4-A, B

The gas line in the upper Mt. Werner Circle box culvert will need to be relocated. Mountain Bell underground cable crosses the existing channel at approximately Station 43+50 and will need to be relocated. Two sanitary sewer manholes are adjacent to the channel at approximately Station 47+25; however, the City of Colorado Springs sanitary sewer atlas indicates that there is no crossing of the existing channel by sanitary sewer at this point.

Reach-Alternative 5-A

There are no significant utility impacts in Reach 5. The proposed berm construction would reduce overhead line clearance by about one foot.

Prioritization of Improvements

Many factors enter into the prioritization of improvements that are not easily quantifiable. Priorities have been established herein for each of the reaches in the project area based strictly on the assessment of flood control needs.

Priority 1 - Reach 2

Reach 2 from upstream of the lower Mt. Werner box culvert (Station 24+88) to the upper reach limits (Station 34+80) has an existing channel with a very low capacity. Residential housing that would experience frequent flood damage is adjacent to both sides of the channel for the full length of Reach 2. The Pebble Creek Apartments which are located near the upstream end of Reach 2 represent the greatest single structure potential for loss of life and damage to be found anywhere within the existing flood area of Spring Run within the project limits. The construction of Reach 2 would provide some reduction in flood damages to the Pebble Creek Apartments prior to the construction of channel improvements for Reach 3.

Priority 2 - Reach 3

Reach 3 channel improvements in combination with Reach 2 channel improvements would provide for 100-year protection to the residential area of Reach 2 and the Pebble Creek Apartments upstream of Reach 2. On this basis Reach 3 is established as the second priority for construction.

Priority 3 - Reach 4

Reach 4 has insufficient capacity for the design discharge throughout the full length of the reach. The apartment complex south of the channel has significant potential for damage along with the residential structures that are adjacent to the channel. The concrete channel immediately upstream of the Upper Mt. Werner Circle culvert is badly deteriorated and the progressive bank sloughing on the south bank of the channel from Station 43+60 to Station 44+10 is presently endangering the house and the apartment complex parking lot on the south side of the channel. There is no question that the progressive channel deterioration and bank sloughing that can be observed in Reach 4 will require immediate short-term maintenance, however, the maintenance would likely consist of stop-gap measures to temporarily stabilize the channel. The channel improvements for Reach 4 are given the third priority, in part, on the basis of providing for permanent stabilization of the channel in Reach 4.

Temporary measures are priority one

Priority 4 - Reach 1

Residential housing that would experience periodic flood damage is adjacent to both sides of the channel for the full length of Reach 1. The existing channel capacity is second only to Reach 5 and therefore flood damage would be less frequent than in Reaches 2 and 3.

Priority 5 - Reach 5

The existing channel in Reach 5 is calculated to have adequate capacity to pass the design discharge of 465 cfs. The berm construction that is proposed for Reach 5 provides a safety factor against channel overtopping due to cross waves that would likely develop in the channel bend with the supercritical flow that exists in Reach 5.

Phasing of Improvements

The phasing of the proposed channel improvement construction for Spring Run involves the consideration of the funding schedule of the City of Colorado Springs and flood control priorities as well as the logical construction sequence. Aside from the flood control priorities which have been identified, the most logical construction sequence would be from downstream (Reach 1) to upstream (Reach 5). The downstream to upstream mode of construction for the entire project would eliminate the need for reduced slope transitions at the downstream end of a construction phase. The need for reduced slope transitions when upstream phases are constructed prior to downstream phases is a direct result of the lowering of the channel invert that is an element of all drainage improvement alternatives from the lower end of Reach 1 to the upper end of Reach 4. The reduced slope transitions at the downstream ends of construction phases would cause a slight reduction in the capacity of the upstream improvements for the interim period prior to construction of the downstream phase. In spite of the problems associated with reduced slope grade transitions from an upstream to a downstream phase, the phasing of construction would best be undertaken in a sequence from highest to lowest priority as outlined in the previous (Prioritization of Improvements) section of this report.

It is noteworthy that the necessity to match the existing invert elevation at the downstream project limits is a constraint in the development of Alternatives for Reaches 1 and 2. This constraint results in a wider channel at a higher elevation than would have been proposed were it not for this constraint.

TABLE VII - 1

PARK MEADOWS/CHEYENNE CREEK
(SPRING RUN)
COST ESTIMATE FOR REACH 1

Alternative A

Item No.	Description	Unit	Unit Price	Est. Quantity	Extension
1	Mobilization	L.S.			$5,210
2	Clearing & Grubbing	L.S.			$6,512
3	Remove Exist. Structures	L.S.			$6,300
4	Control of Water	L.S.			$7,814
5	Structure Excavation	C.Y.	$4.00	1,933	$7,732
6	Structure Backfill	C.Y.	$3.00	479	$1,437
7	Reinforced Concrete	C.Y.	$325.00	349	$113,425
8	Prefabricated Plastic Drain Material	S.Y.	$15.00	255	$3,825
9	Class A Filter Material	C.Y.	$22.00	135	$2,970
10	Native Grass Seeding	S.F.	$ 0.08	10,640	$851

SUBTOTAL =		$156,076
Engr design & const management (15%)	=	$23,411
Contingencies (10%)	=	$15,608
TOTAL	=	$195,095
(AVG COST/L.F.)	=	$513

TABLE VII-2

PARK MEADOWS/CHEYENNE CREEK
(SPRING RUN)
COST ESTIMATE FOR REACH 2

Item No.	Description	Unit	Unit Price	Alternative A Est. Quantity	Alternative A Extension	Alternative B Est. Quantity	Alternative B Extension	Alternative C Est. Quantity	Alternative C Extension
1	Mobilization	L.S.			$21,776		$15,502		$15,453
2	Clearing & Grubbing	L.S.			$27,221		$19,377		$19,316
3	Remove Exist. Structures	L.S.			$2,200		$8,700		$8,700
4	Control of Water	L.S.			$32,665		$23,253		$23,180
5	Structure Excavation	C.Y.	$4.00	5,944	$23,776	77	$308	77	$308
6	Structure Backfill	C.Y.	$3.00	1,458	$4,374	5	$15	5	$15
7	Reinforced Concrete	C.Y.	$325.00	934	$303,550	0	$0	0	$0
8	Prefabricated Plastic Drain Material	S.Y.	$15.00	7,342	$110,130	27	$405	25	$375
9	Class A Filter Material	C.Y.	$22.00	418	$9,196	3	$66	2	$44
10	60-inch RCP, Class 3	L.F.	$128.00	0	$0	2,340	$299,520	0	$0
11	Pre-cast Conc 8x5 RCB	L.F.	$255.00	0	$0	0	$0	1,170	$298,350
12	Transition Structure Sta 24+30 to 24+38 and Sta 24+88 to 24+96	C.Y.	$325.00	13	$4,225	0	$0	0	$0
13	Transition Struct & Hdwl Sta 34+70 to 34+80	C.Y.	$325.00	0	$0	55	$17,875	55	$17,875
14	Reinf Conc Drop Struct Sta 24+30,30+50,31+50	C.Y.	$325.00	104	$33,800	0	$0	0	$0
15	Relocate Exist Gas Line (Sta 24+63)	L.S.			$10,000		$10,000		$10,000
16	Relocate Exist Water Line (Sta 24+53)	L.S.	$0		$0		$10,000		$10,000
17	Relocate Exist Telephone Cable	L.F.	$5.00	2,000	$10,000	2,000	$10,000	2,000	$10,000

Item No.	Description	Unit	Unit Price	Est. Quantity	Extension	Est. Quantity	Extension	Est. Quantity	Extension
18	Relocate Exist Television Cable	L.F.	$5.00	2,000	$10,000	2,000	$10,000	2,000	$10,000
19	Furnish/Install 42-inch Chain Link Fence	L.F.	$8.00	2,220	$17,760	2,220	$17,760	2,220	$17,760
20	ROW Restoration	L.S.			$5,000		$5,000		$5,000
21	Native Grass Seeding	S.F.	$0.08	20,000	$1,600	20,000	$1,600	20,000	$1,600
22	Traffic Control	L.S.			$1,000		$5,000		$5,000

SUBTOTAL - ALT A =	$628,273	SUBTOTAL - ALT B =	$454,381	SUBTOTAL - ALT C =	$452,976
Engr design & const management (15%) =	$94,241	Engr design & const management (15%) =	$68,157	Engr design & const management (15%) =	$67,946
Contingencies(10%)=	$62,827	Contingencies(10%)=	$45,438	Contingencies(10%)=	$45,298
TOTAL -ALT A =	$785,341	TOTAL -ALT B =	$567,977	TOTAL -ALT C =	$566,220
(AVG COST/L.F.)=	$677	(AVG COST/L.F.)=	$490	(AVG COST/L.F.)=	$488

TABLE VII-3

PARK MEADOWS/CHEYENNE CREEK
(SPRING RUN)
COST ESTIMATE FOR REACH 3

Item No.	Description	Unit	Unit Price	Alternative A Est. Quantity	Alternative A Extension	Alternative B Est. Quantity	Alternative B Extension
1	Mobilization	L.S.			$10,616		$7,352
2	Clearing & Grubbing	L.S.			$13,269		$9,189
3	Remove Exist. Structures	L.S.			$1,250		$1,250
4	Control of Water	L.S.			$15,923		$11,027
5	Low Flow Channel	L.F.	$40.00	730	$29,200	730	$29,200
6	Unclassified Exc. Used for Fill	C.Y.	$4.00	3000	$12,000	3000	$12,000
7	Transition Struct & Wingwall, Sta. 41+77 to 41+92	C.Y.	$325.00	20	$6,500	20	$6,500
8-a	Soil Prep. and Sodding	S.F.	$0.25	160000	$40,000		
8-b	Soil Prep. and Bluegrass Seeding	S.F.	$0.10			160000	$16,000
9	Low Flow Channel Bridge	L.S.			$2,000		$2,000
10	Low Flow Channel Weir Structure	L.S.			$4,000		$4,000
11	Reinforced Concrete for Drop Structure	C.Y.	$325.00	75	$24,375	75	$24,375
12	Plantings	L.S.			$40,000		$10,000
13	Repl. Irrigation System	S.F.	$0.20	260000	$52,000	260000	$52,000
14	Rel. Softball Field (New Backstop)	L.S.			$5,575		$5,575

Item No.	Description	Unit	Unit Price	Est. Quantity	Extension	Est. Quantity	Extension
15	Sidewalk Removal	S.F.	$0.60	2230	$1,338	2230	$1,338
16-a	Conc. Maintenance Path	S.F.	$3.50	12000	$42,000		
16-b	Asph. Maintenance Path	S.F.	$1.30			12000	$15,600
17	New Light Fixtures	EA.	$1,200	2	$2,400	1	$1,200
18	Relocate Exist Telephone Cable	L.F.	$5.00	800	$4,000	800	$4,000

SUBTOTAL - ALT A =	$306,446	SUBTOTAL - ALT A-1	$212,606
Engr design & const management (15%) =	$45,967	Engr design & const management (15%) =	$31,891
Contingencies(10%)=	$30,645	Contingencies(10%)=	$21,261
TOTAL - ALT A =	$383,058	TOTAL - ALT A-1 =	$265,758
(AVG COST/L.F.) =	$460	(AVG COST/L.F.) =	$319
(AVG COST/S.F.) =	$1.47	(AVG COST/S.F.) =	$1.02

TABLE VII-4

PARK MEADOWS/CHEYENNE CREEK
(SPRING RUN)
COST ESTIMATE FOR REACH 4

Item No.	Description	Unit	Unit Price	Alternative A Est. Quantity	Alternative A Extension	Alternative B Est. Quantity	Alternative B Extension
1	Mobilization	L.S.			$7,531		$9,370
2	Clearing & Grubbing	L.S.			$9,414		$11,713
3	Remove Exist. Structures	L.S.			$2,000		$2,000
4	Control of Water	L.S.			$11,297		$14,055
5	Structure Excavation	C.Y.	$4.00	2,243	$8,972	3,207	$12,828
6	Structure Backfill	C.Y.	$3.00	819	$2,457	832	$2,496
7	Reinforced Concrete	C.Y.	$325.00	302	$98,150	413	$134,225
8	Prefabricated Plastic Drain Material	S.Y.	$15.00	422	$6,330	422	$6,330
9	Class A Filter Material	C.Y.	$22.00	134	$2,948	200	$4,400
10	Transition Structure Sta 42+41 to 42+75	C.Y.	$325.00	27	$8,775	27	$8,775
11	Reinf Conc Drop Struct Sta 43+30,44+00,46+90	C.Y.	$325.00	90	$29,250	104	$33,800
12	Reinf Conc Foot Bridge	L.S.			$4,000		$4,000
13	Relocate Exist Gas Line (Sta 42+07)	L.S.			$10,000		$10,000
14	Relocate Exist Telephone Cable Crossing Chnl	L.S.			$5,000		$5,000
15	Furnish/Install 42-inch Chain Link Fence	L.F.	$8.00	1,136	$9,088	1,136	$9,088
16	ROW Restoration	L.S.			$2,500		$2,500

Item No.	Description	Unit	Unit Price	Est. Quantity	Extension		Est. Quantity	Extension
17	Native Grass Seeding	S.F.	$0.08	10,200	$816		10,200	$816
			SUBTOTAL - ALT A =		$218,529	SUBTOTAL - ALT B =		$271,397
			Engr design & const management (15%) =		$32,779	Engr design & const management (15%) =		$40,710
			Contingencies(10%)=		$21,853	Contingencies(10%)=		$27,140
			TOTAL -ALT A =		$273,161	TOTAL -ALT B =		$339,246
			(AVG COST/L.F.)=		$442	(AVG COST/L.F.)=		$549

TABLE VII-5

PARK MEADOWS/CHEYENNE CREEK
(SPRING RUN)
COST ESTIMATE FOR REACH 5

Alternative A

Item No.	Description	Unit	Unit Price	Est. Quantity	Extension
1	Mobilization	L.S.			$205
2	Clearing & Grubbing	L.S.			$256
3	Embankment Fill	C.Y.	$6.00	135	$810
4	Type M Riprap	C.Y.	$40.00	65	$2,600
5	Class A Filter Material	C.Y.	$22.00	35	$770
6	Filter Cloth	S.Y.	$4.00	200	$800
7	Native Grass Seeding	S.F.	$0.08	1800	$144
			SUBTOTAL - ALT A =		$5,585
			Engr design & const management (15%) =		$838
			Contingencies(10%)=		$559
			TOTAL - ALT A =		$6,981
			(AVG COST/L.F.) =		$37

REFERENCES

1. Lincoln-DeVore Testing Laboratory, Inc., "Engineering Study of Southwest Area Drainage Basin (Cheyenne Creek, Cheyenne Run, and Spring Run) Colorado Springs, Colorado", February, 1984.

2. United States Army Corps of Engineers, HEC-2 Water Surface Profiles Program Users Manual, August, 1979.

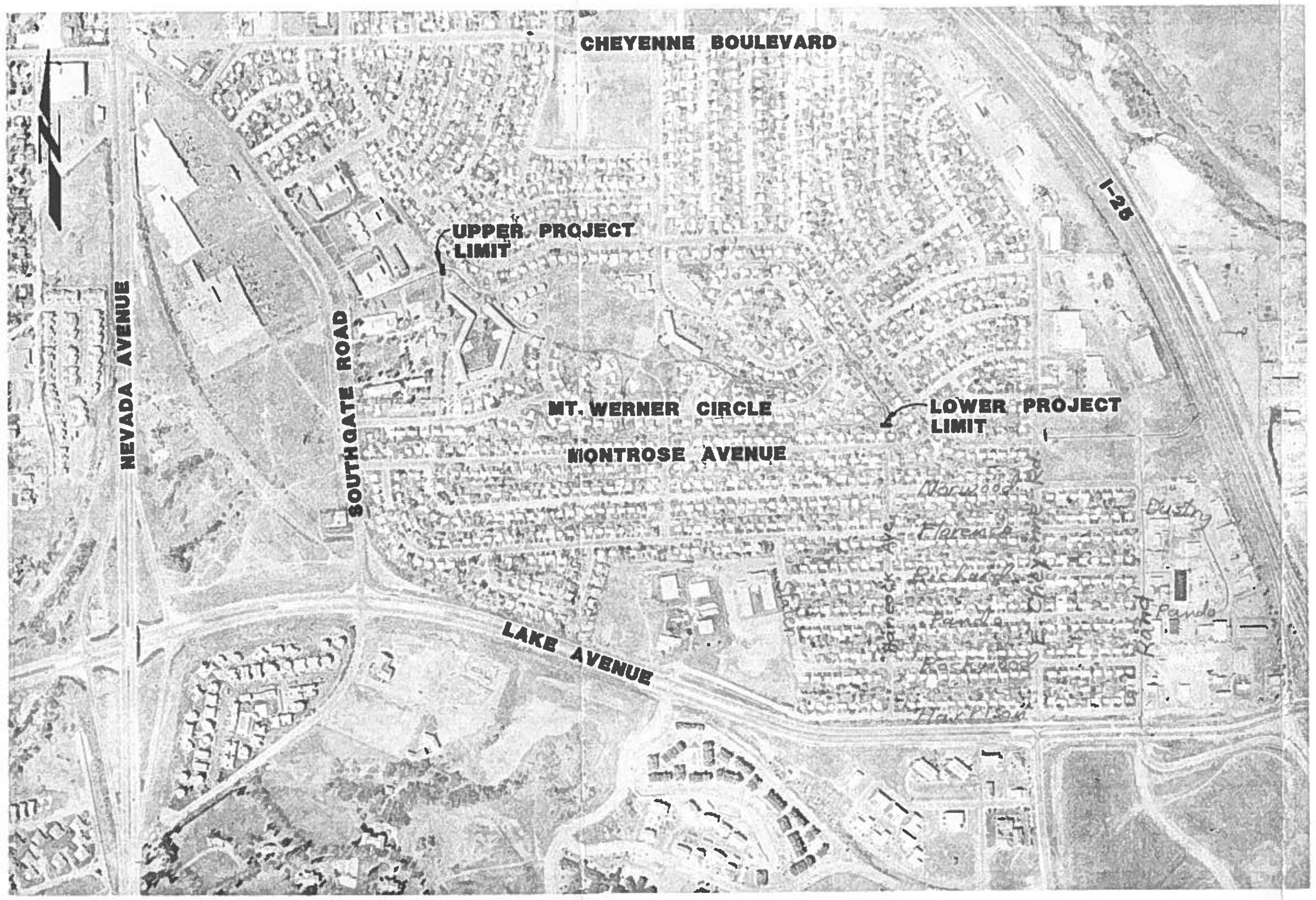
3. Ven Te Chow, Open Channel Hydraulics, McGraw-Hill, 1959.

4. "Urban Storm Drainage Criteria Manual", Denver Regional Council of Governments (prepared by Wright-McLaughlin Engineers), March, 1969.

APPENDIX A

DRAWINGS

APPENDIX A
PARK MEADOWS/ CHEYENNE CREEK (SPRING RUN) DRAINAGE IMPROVEMENTS PRELIMINARY DESIGN



INDEX OF SHEETS

SHEET	DESCRIPTION
1	COVER SHEET
2	ALTERNATIVE 1-A, STA. 19+20 TO STA. 23+00
3	ALTERNATIVE 2-A, STA. 23+00 TO STA. 27+00
4	ALTERNATIVE 2-A, STA. 27+00 TO STA. 33+00
5	ALTERNATIVE 2-A, STA. 33+00 TO STA. 34+80
6	ALTERNATIVES 2-B & 2-C, STA. 23+00 TO STA. 27+00
7	ALTERNATIVES 2-B & 2-C, STA. 27+00 TO STA. 33+00
8	ALTERNATIVES 2-B & 2-C, STA. 33+00 TO STA. 34+80
9	ALTERNATIVES 3-A & 3-B, STA. 34+80 TO STA. 39+00
10	ALTERNATIVES 3-A & 3-B, STA. 39+00 TO STA. 41+92
11	ALTERNATIVF 4-A, STA. 41+92 TO STA. 45+00
12	ALTERNATIVE 4-A, STA. 45+00 TO STA. 48+10
13	ALTERNATIVE 4-B, STA. 41+92 TO STA. 45+00
14	ALTERNATIVE 4-R, STA. 45+00 TO STA. 48+10
15	ALTERNATIVE 5-A, STA. 48+10 TO STA. 50+00
16	REACH 3 - MASTER PLAN DEVELOPMENT

CITY OF
COLORADO SPRINGS

SPRING RUN TRIBUTARY TO FOUNTAIN CREEK

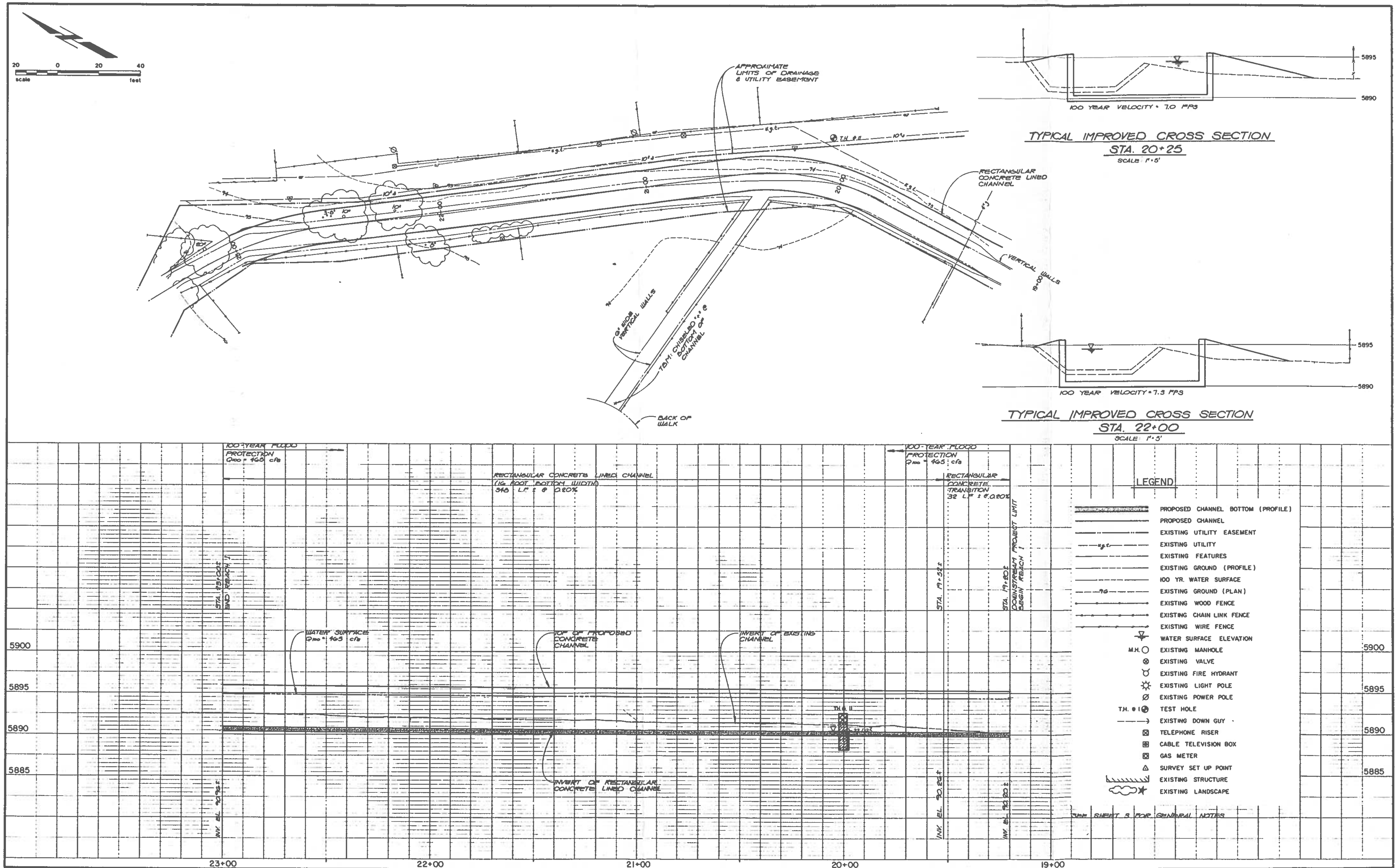
400 0 400 800
scale 1"=400' feet

CONSULTING
ENGINEERS

SELLARDS & GRIGG, INC.
LAKEWOOD, COLORADO

S & G JOB NO. 85433
SHEET 1 OF 16

COLORADO REGISTERED PROFESSIONAL ENGINEER 17191



LEGEND

- PROPOSED CHANNEL BOTTOM (PROFILE)
- PROPOSED CHANNEL
- EXISTING UTILITY EASEMENT
- EXISTING UTILITY
- EXISTING FEATURES
- EXISTING GROUND (PROFILE)
- 100 YR. WATER SURFACE
- EXISTING GROUND (PLAN)
- EXISTING WOOD FENCE
- EXISTING CHAIN LINK FENCE
- EXISTING WIRE FENCE
- WATER SURFACE ELEVATION
- M.H. EXISTING MANHOLE
- EXISTING VALVE
- EXISTING FIRE HYDRANT
- EXISTING LIGHT POLE
- EXISTING POWER POLE
- T.H. # 1 TEST HOLE
- EXISTING DOWN GUY
- TELEPHONE RISER
- CABLE TELEVISION BOX
- GAS METER
- SURVEY SET UP POINT
- EXISTING STRUCTURE
- EXISTING LANDSCAPE

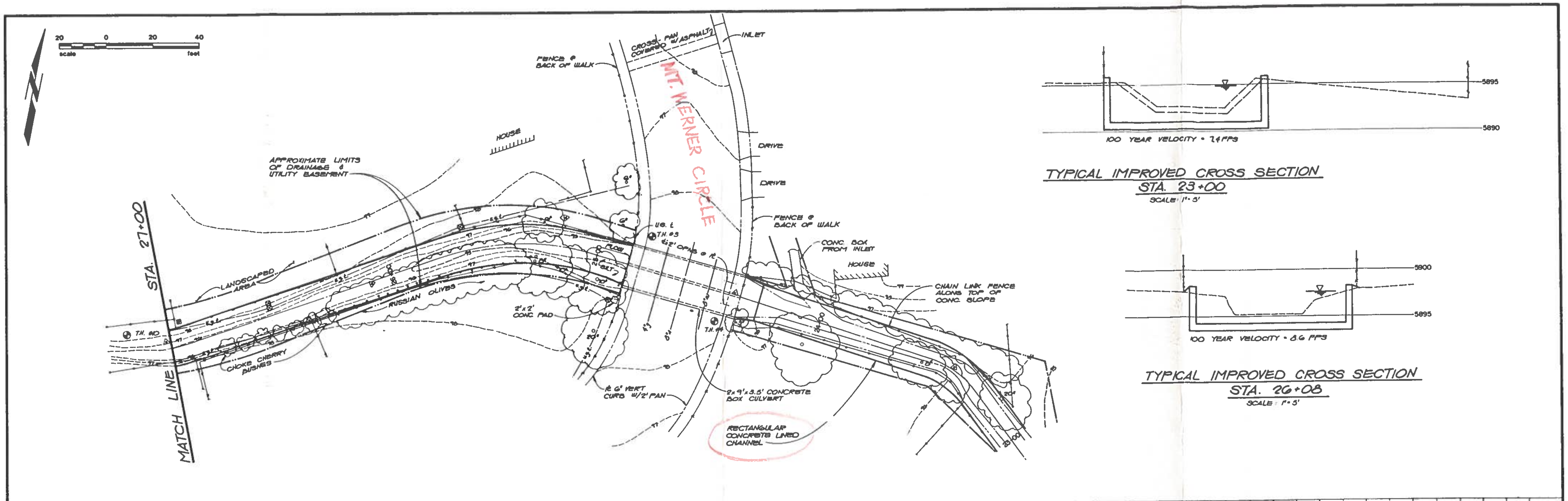
SEE SHEET 3 FOR GENERAL NOTES

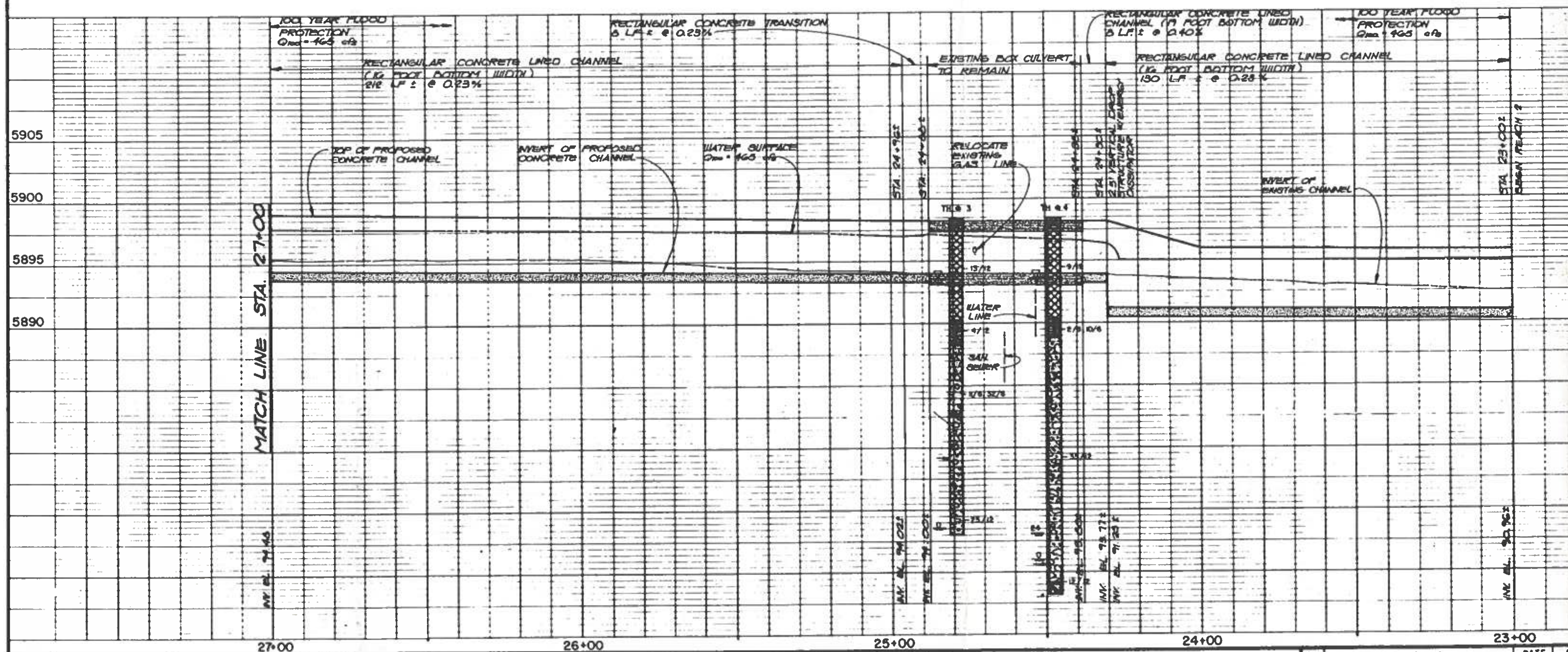
NO.	REVISIONS	DATE	BY

CONSULTING ENGINEERS SELLARDS & GRIGG, Inc.
LAKEWOOD, COLORADO

DESIGNED BY CAM	CITY OF COLORADO SPRINGS	85433
DRAWN BY JAF	PARK MEADOWS / CHEYENNE CREEK (SPRING RUN)	
CHECKED BY TGF	ALTERNATIVE I-A	AS NOTED
OCT. 1985	STA. 19+20 TO STA. 23+00	2 OF 16

COLORADO REGISTERED ... 17192





GENERAL NOTES

1. ALL MAPPING BASED ON LAND SURVEY BY SELLARDS & GRIGG, INC., AUGUST 5-9, 1985.
2. TEMPORARY BENCHMARK (TBM): BONNET BOLT ON FIRE HYDRANT AT MT. WERNER CIRCLE CUL-DE-SAC, ELEVATION 5919.85, PER LEE WHITEHEAD & ASSOCIATES, AUGUST 20, 1985.
3. UTILITY AND EASEMENT LOCATIONS SHOWN ARE APPROXIMATE. ACTUAL LOCATIONS TO BE DETERMINED IN FINAL DESIGN.
4. TELEVISION CABLES (NOT SHOWN ON DRAWINGS) FOLLOW APPROXIMATELY SAME ALIGNMENT AS UNDERGROUND TELEPHONE CABLES.
5. ALL CROSS SECTIONS SHOWN LOOKING UPSTREAM.

NO.	REVISIONS	DATE	BY

COLORADO REGISTERED PROFESSIONAL ENGINEER CHARLES A. McKNIGHT 17191

CONSULTING ENGINEERS SELLARDS & GRIGG, Inc. LAKEWOOD COLORADO

TGF
JAF
CAM
OCT. 1985

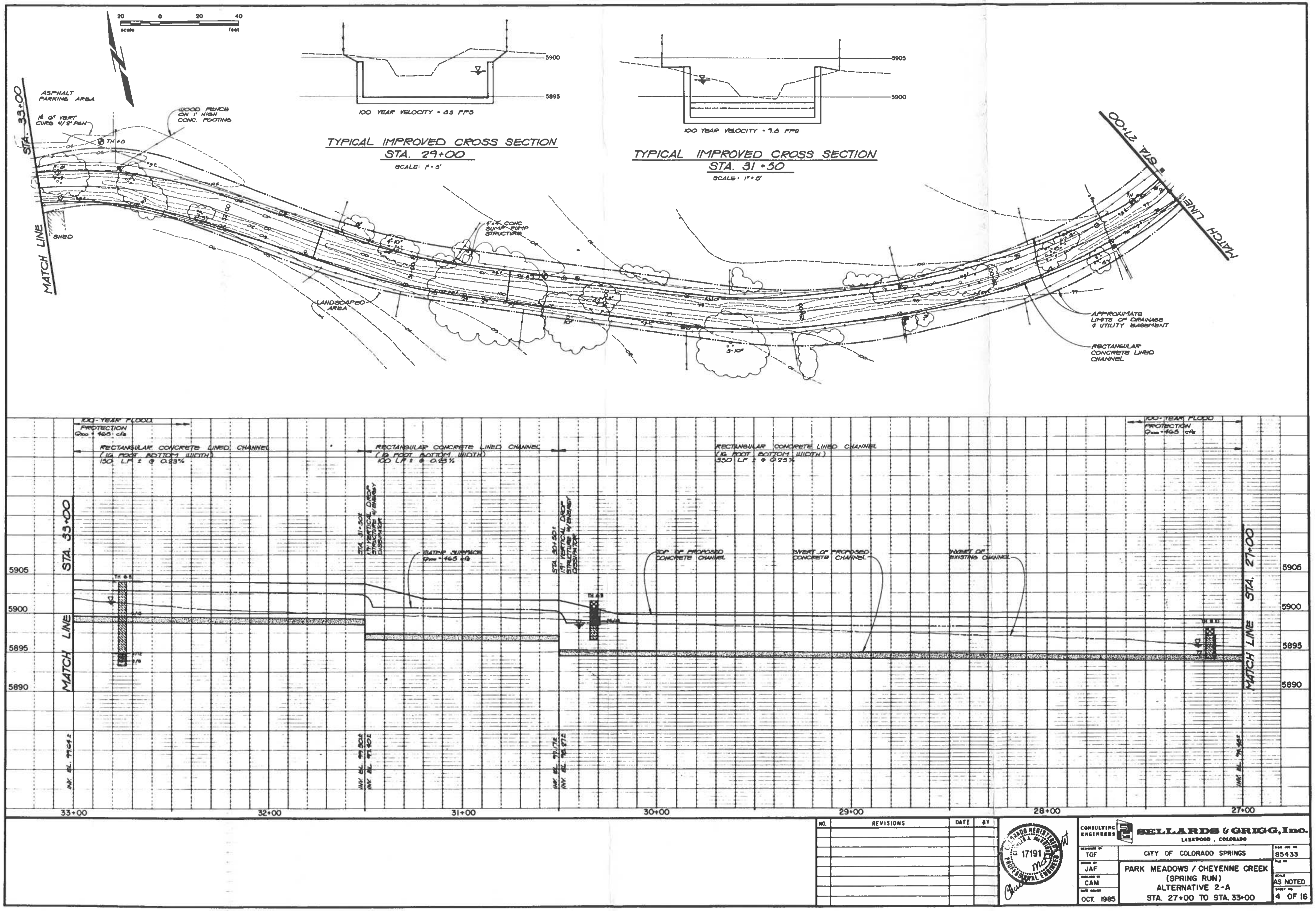
CITY OF COLORADO SPRINGS

PARK MEADOWS / CHEYENNE CREEK
(SPRING RUN)
ALTERNATIVE 2-A
STA. 23+00 TO STA. 27+00

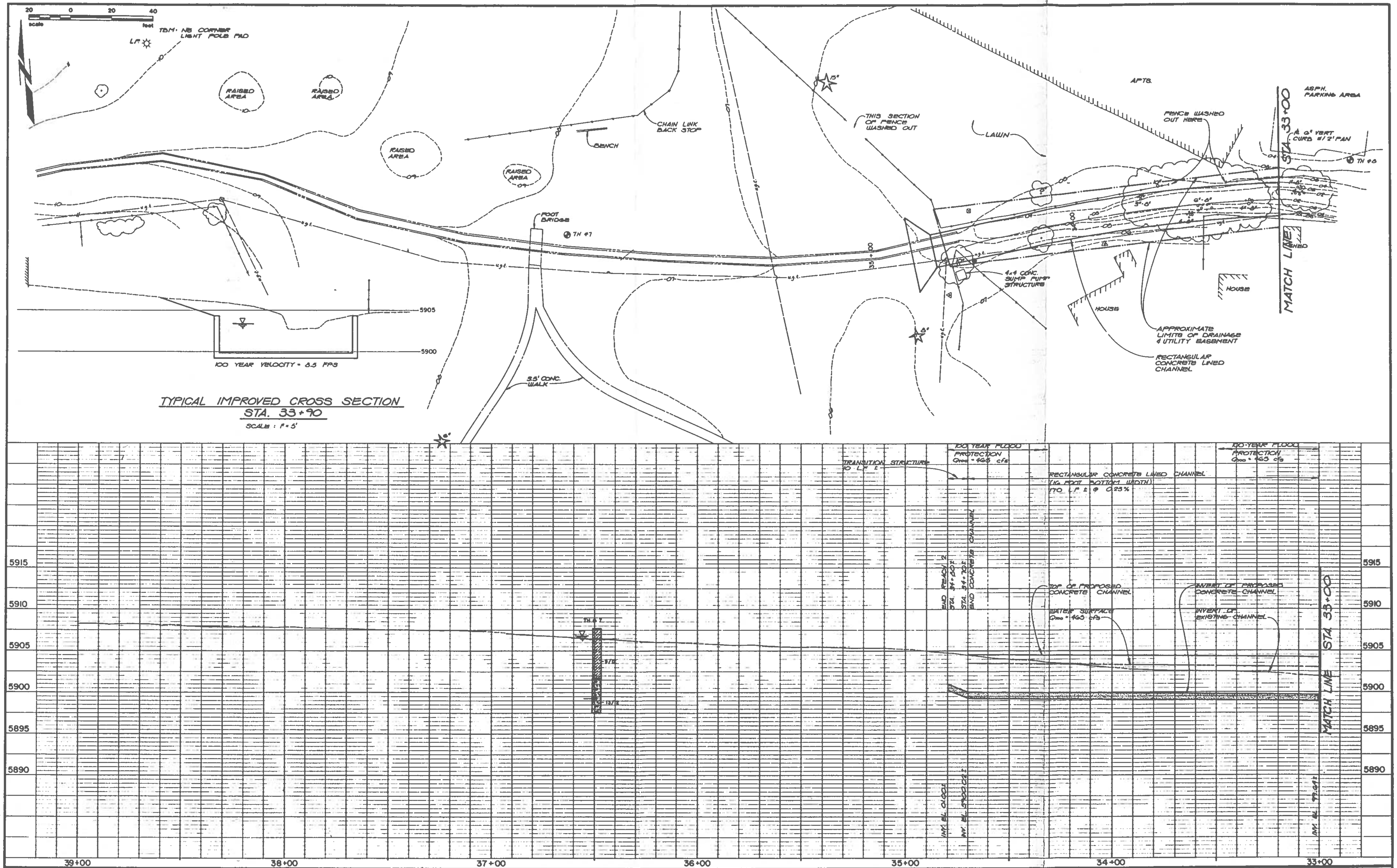
85433

AS NOTED

3 OF 16



20 0 20 40
scale feet
ASPHALT PARKING AREA
WOOD FENCE ON 1' HIGH CONC. FOOTING
TH #8
SHED
MATCH LINE STA. 33+00
MATCH LINE STA. 27+00
100 YEAR VELOCITY = 8.5 FPS
TYPICAL IMPROVED CROSS SECTION
STA. 29+00
SCALE: 1" = 5'
100 YEAR VELOCITY = 7.8 FPS
TYPICAL IMPROVED CROSS SECTION
STA. 31+50
SCALE: 1" = 5'
5900
5895
5905
CONC SUMP PUMP STRUCTURE
LANDSCAPED AREA
APPROXIMATE LIMITS OF DRAINAGE & UTILITY EASEMENT
RECTANGULAR CONCRETE LINED CHANNEL
100-YEAR FLOOD PROTECTION Q100 = 465 cfs
RECTANGULAR CONCRETE LINED CHANNEL (16 FOOT BOTTOM WIDTH) 150 LF ± @ 0.23%
RECTANGULAR CONCRETE LINED CHANNEL (16 FOOT BOTTOM WIDTH) 100 LF ± @ 0.23%
RECTANGULAR CONCRETE LINED CHANNEL (16 FOOT BOTTOM WIDTH) 350 LF ± @ 0.23%
WATER SURFACE Q100 = 465 cfs
TOP OF PROPOSED CONCRETE CHANNEL
INVERT OF PROPOSED CONCRETE CHANNEL
INVERT OF EXISTING CHANNEL
5905
5900
5895
5890
33+00
32+00
31+00
30+00
29+00
28+00
27+00
NO.
REVISIONS
DATE
BY
17191
CONSULTING ENGINEERS
SELLARDS & GRIGG, Inc.
LAKEWOOD, COLORADO
TGF
JAF
CAM
OCT. 1985
CITY OF COLORADO SPRINGS
PARK MEADOWS / CHEYENNE CREEK
(SPRING RUN)
ALTERNATIVE 2-A
STA. 27+00 TO STA. 33+00
85433
AS NOTED
4 OF 16



NO.	REVISIONS	DATE	BY

COLORADO REGISTERED PROFESSIONAL ENGINEER 17191

CONSULTING ENGINEERS
SELLARDS & GRIGG, Inc.
LAKEWOOD, COLORADO

TGF	CITY OF COLORADO SPRINGS	85433
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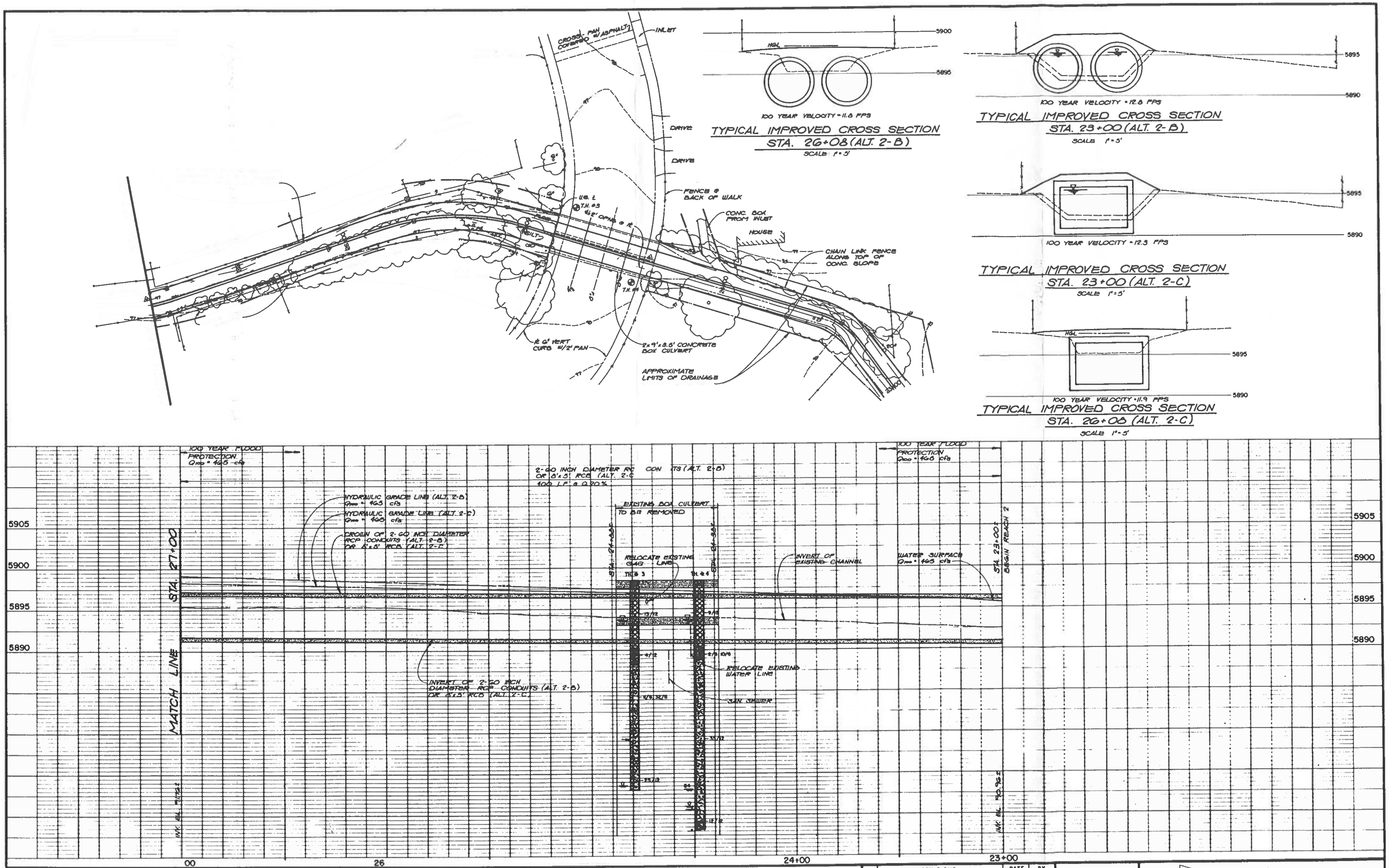
OCT. 1985		5 OF 16

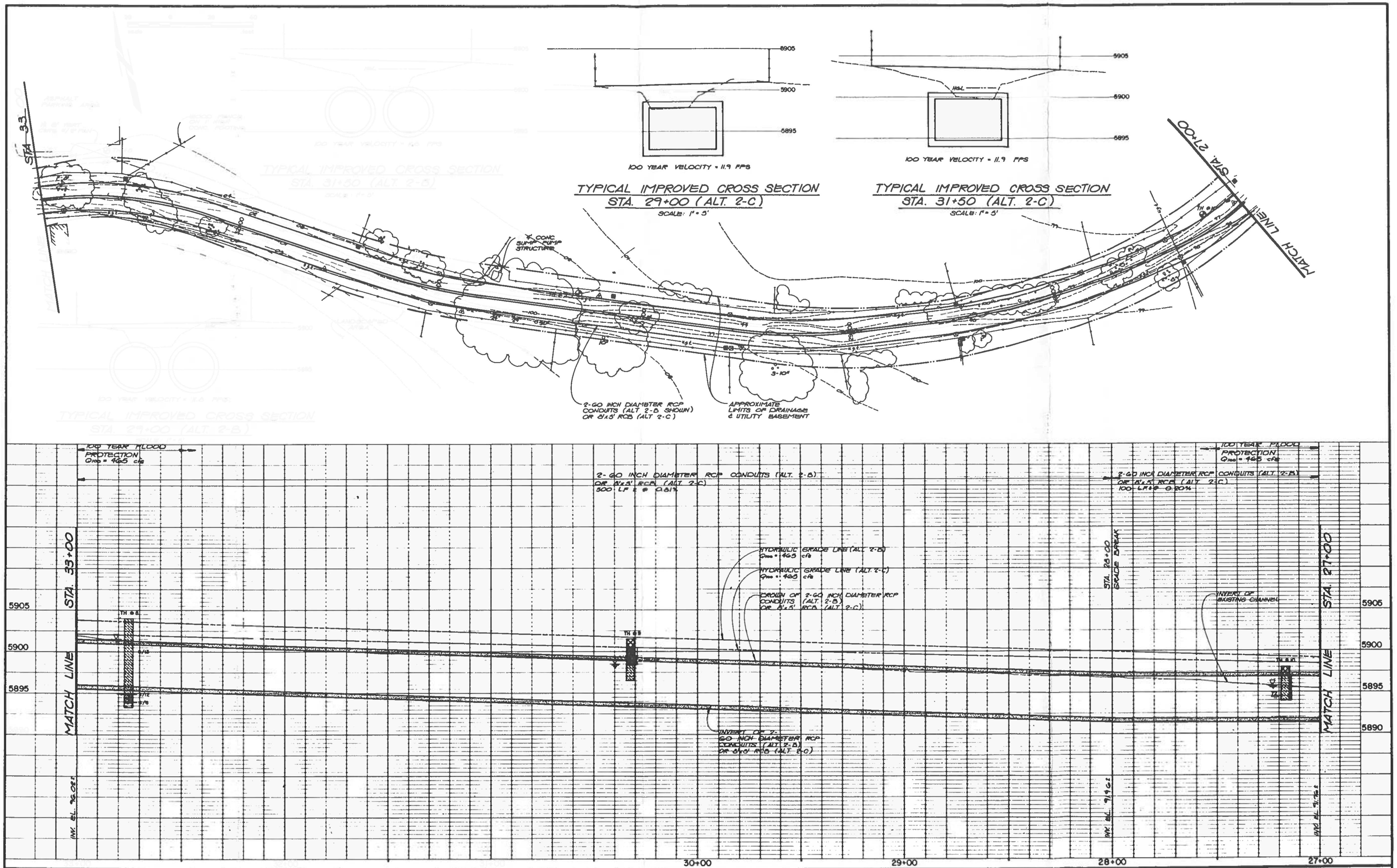


NO.	REVISIONS	DATE	BY

COLORADO REGISTERED PROFESSIONAL ENGINEER 17191

CONSULTING ENGINEERS SELLARDS & GRIGG, Inc. LAKEWOOD, COLORADO

DESIGNED BY TGF	CITY OF COLORADO SPRINGS	JOB NO. 85433
DRAWN BY JAF	PARK MEADOWS / CHEYENNE CREEK (SPRING RUN) ALTERNATIVES 2-B & 2-C STA. 23+00 TO STA. 27+00	FILE NO.
CHECKED BY CAM		SCALE AS NOTED
DATE ISSUED OCT. 1985		SHEET 6 OF 16



NO.	REVISIONS	DATE	BY

COLORADO REGISTERED PROFESSIONAL ENGINEER CHARLES A. McKNIGHT 17191

CONSULTING ENGINEERS **SELLARDS & GRIGG, Inc.** LAKEWOOD, COLORADO

TGF	CITY OF COLORADO SPRINGS	85433
JAF	PARK MEADOWS / CHEYENNE CREEK (SPRING RUN) ALTERNATIVES 2-B & 2-C STA. 27+00 TO STA. 33+00	
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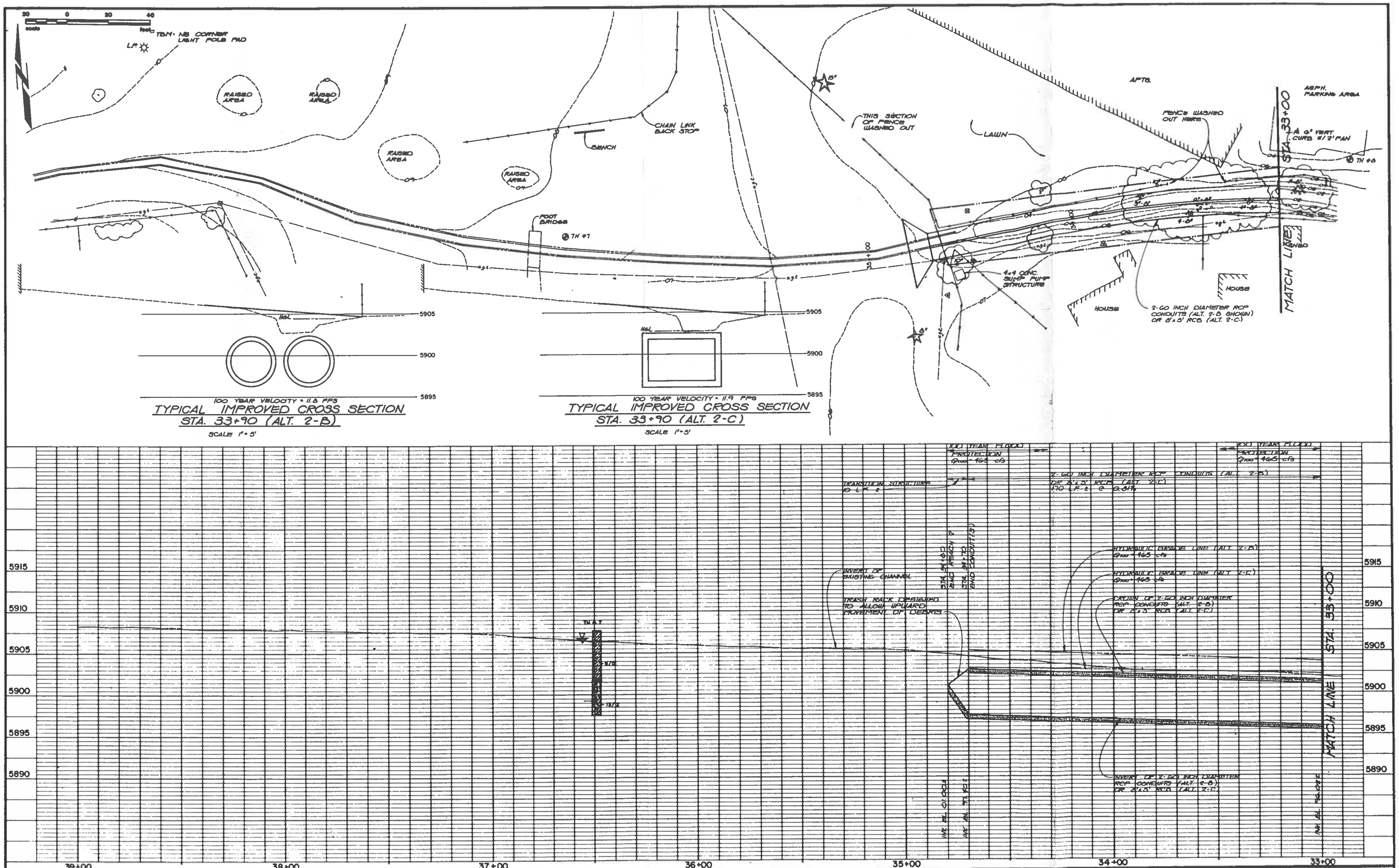
OCT. 1985		7 OF 16



NO.	REVISIONS	DATE	BY

CONSULTING ENGINEERS SELLARDS & GRIGG, Inc.
LAKEWOOD, COLORADO

COLORADO REGISTERED PROFESSIONAL ENGINEER 17197

DESIGNED BY TGF	CITY OF COLORADO SPRINGS	85433
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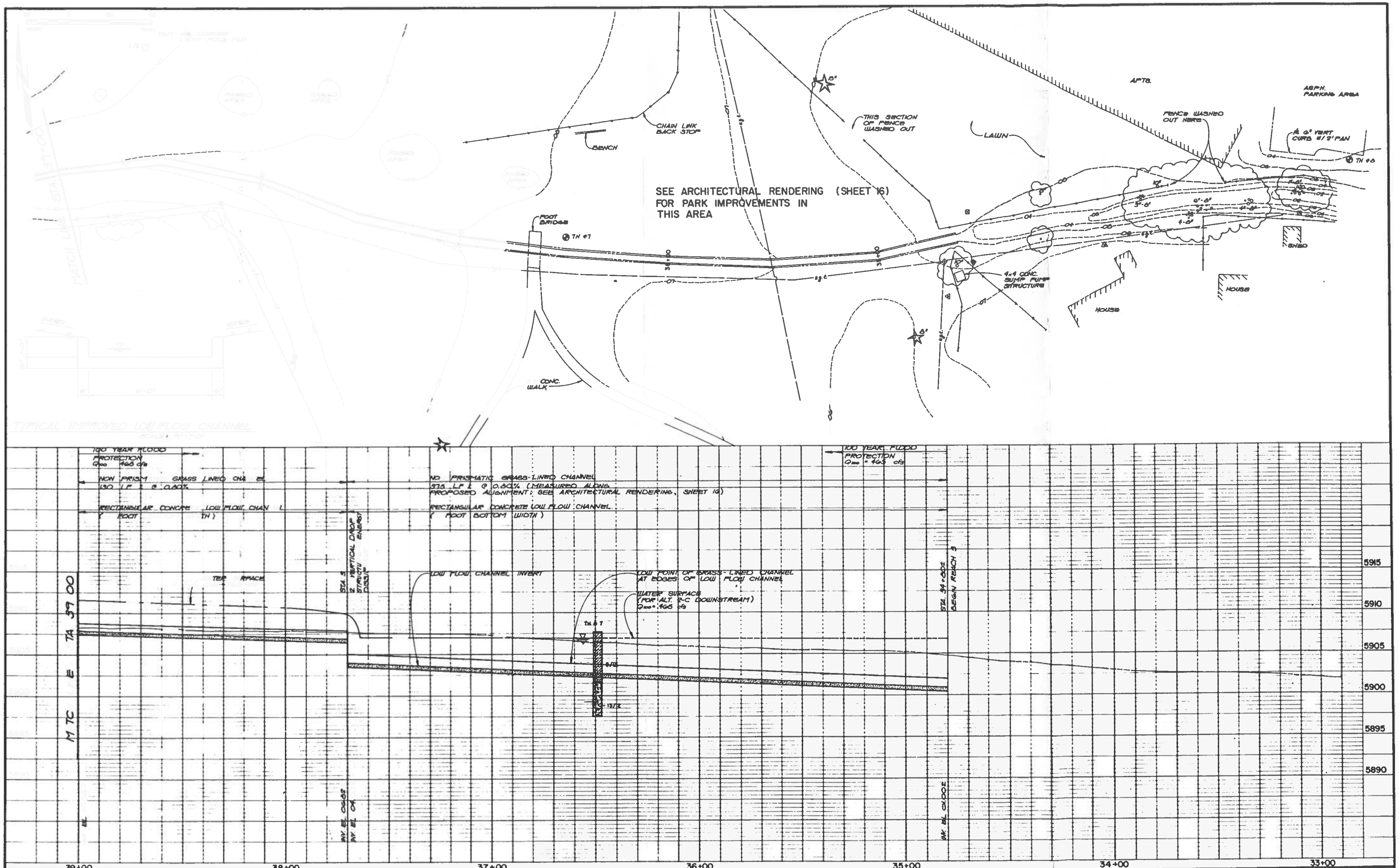
CHECKED BY CAM		SCALE AS NOTED
OCT. 1985		8 OF 16



NO.	REVISIONS	DATE	BY

COLORADO REGISTERED PROFESSIONAL ENGINEER CHARLES A. McKNIGHT 17191

CONSULTING ENGINEERS
SELLARDS & GRIGG, Inc.
LAKEWOOD, COLORADO

CAM	CITY OF COLORADO SPRINGS	85433
JAF	PARK MEADOWS / CHEYENNE CREEK (SPRING RUN) ALTERNATIVES 3-A & 3-B STA. 34+80 TO STA. 39+00	
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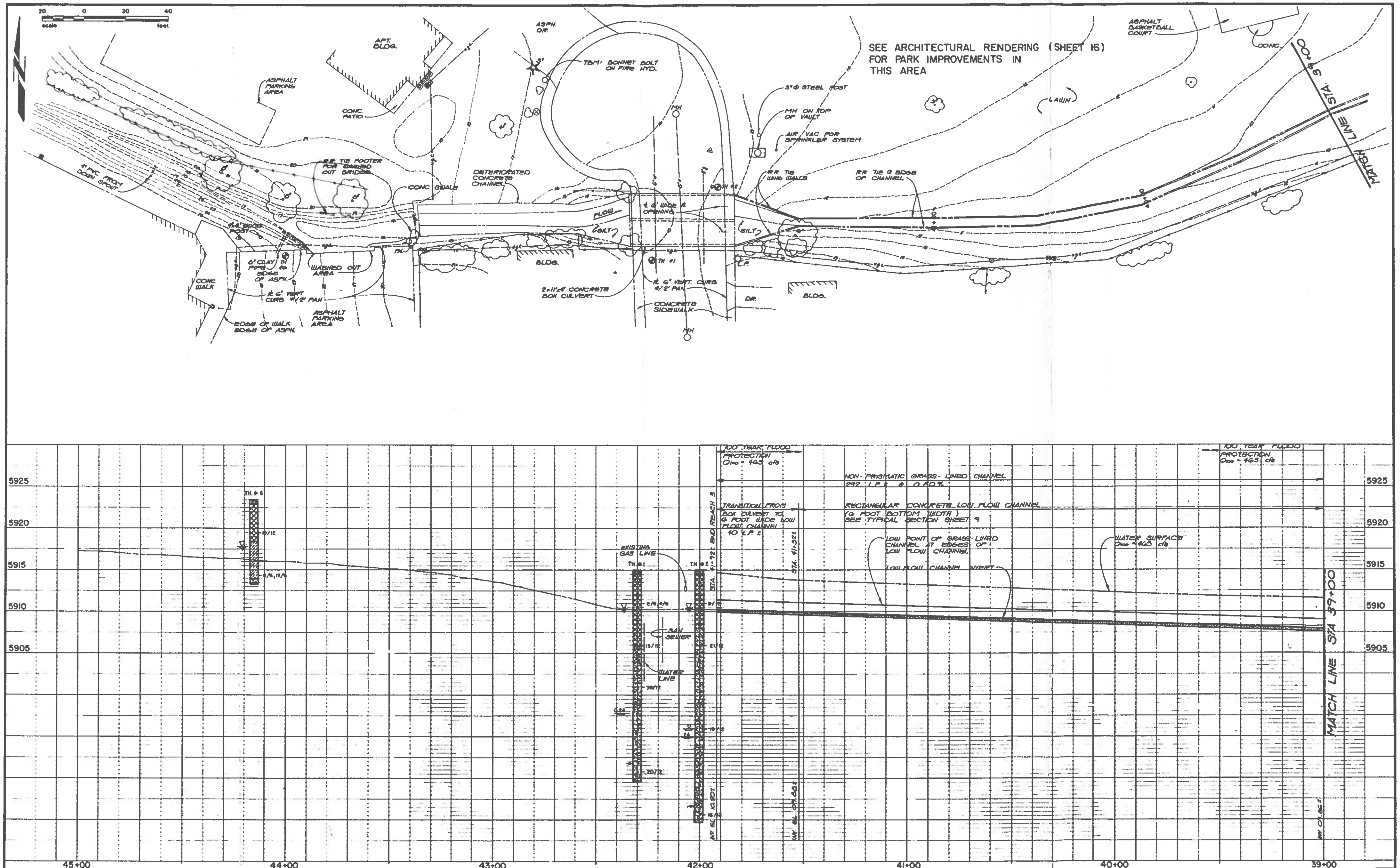
OCT. 1985		9 OF 16

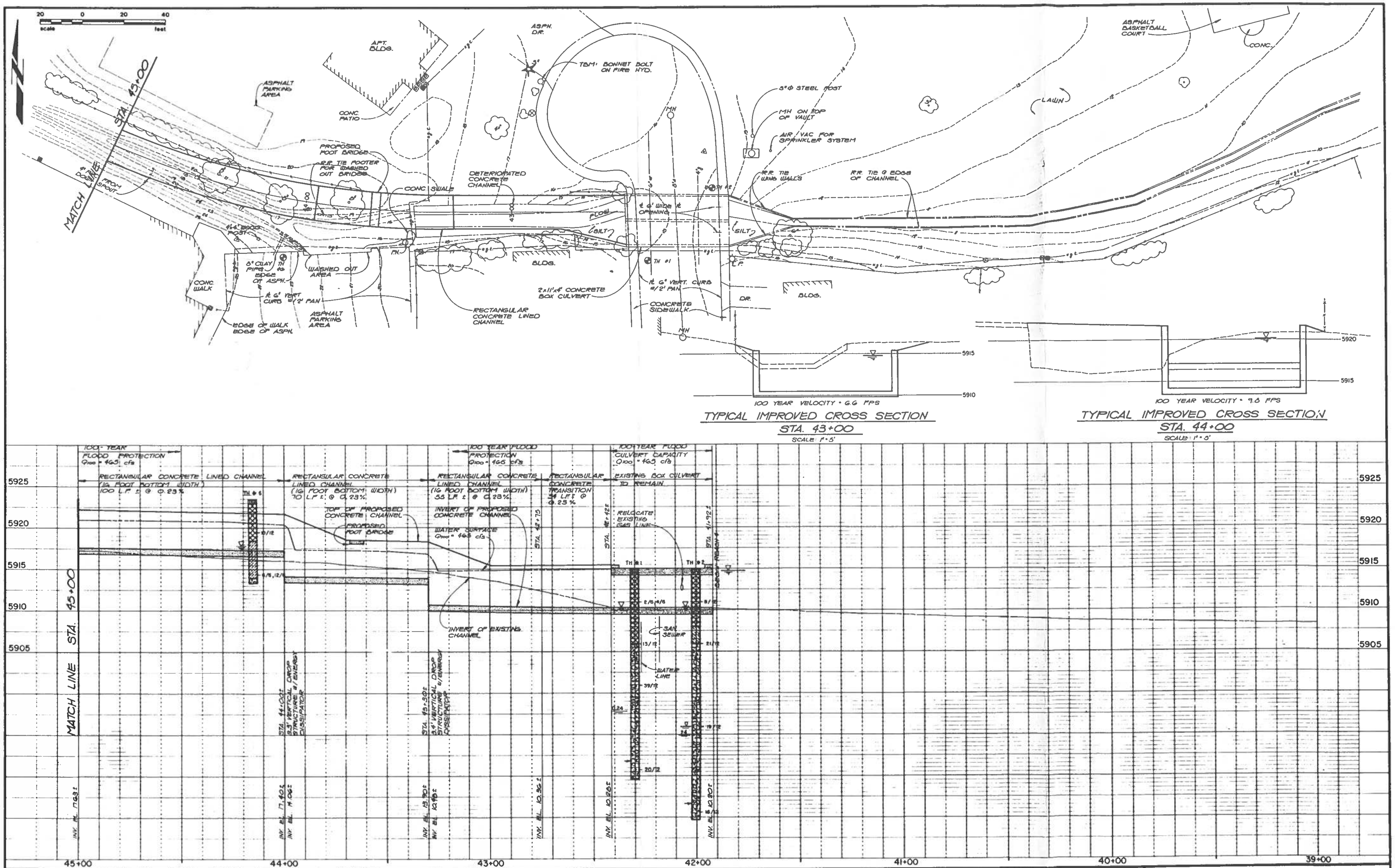


NO.	REVISIONS	DATE	BY

COLORADO REGISTERED PROFESSIONAL ENGINEER CHARLES A. McKNIGHT 17191

CONSULTING ENGINEERS SELLARDS & GRIGG, Inc. LAKEWOOD, COLORADO

DESIGNED BY CAM	CITY OF COLORADO SPRINGS	JOB NO. 85433
DRAWN BY JAF	PARK MEADOWS / CHEYENNE CREEK (SPRING RUN) ALTERNATIVES 3-A & 3-B STA. 39+00 TO STA. 41+92	FILE NO.
CHECKED BY TGF		SCALE AS NOTED
DATE ISSUED OCT. 1985		SHEET NO. 10 OF 16



NO.	REVISIONS	DATE	BY

CONSULTING ENGINEERS SELLARDS & GRIGG, Inc.
LAKEWOOD, COLORADO

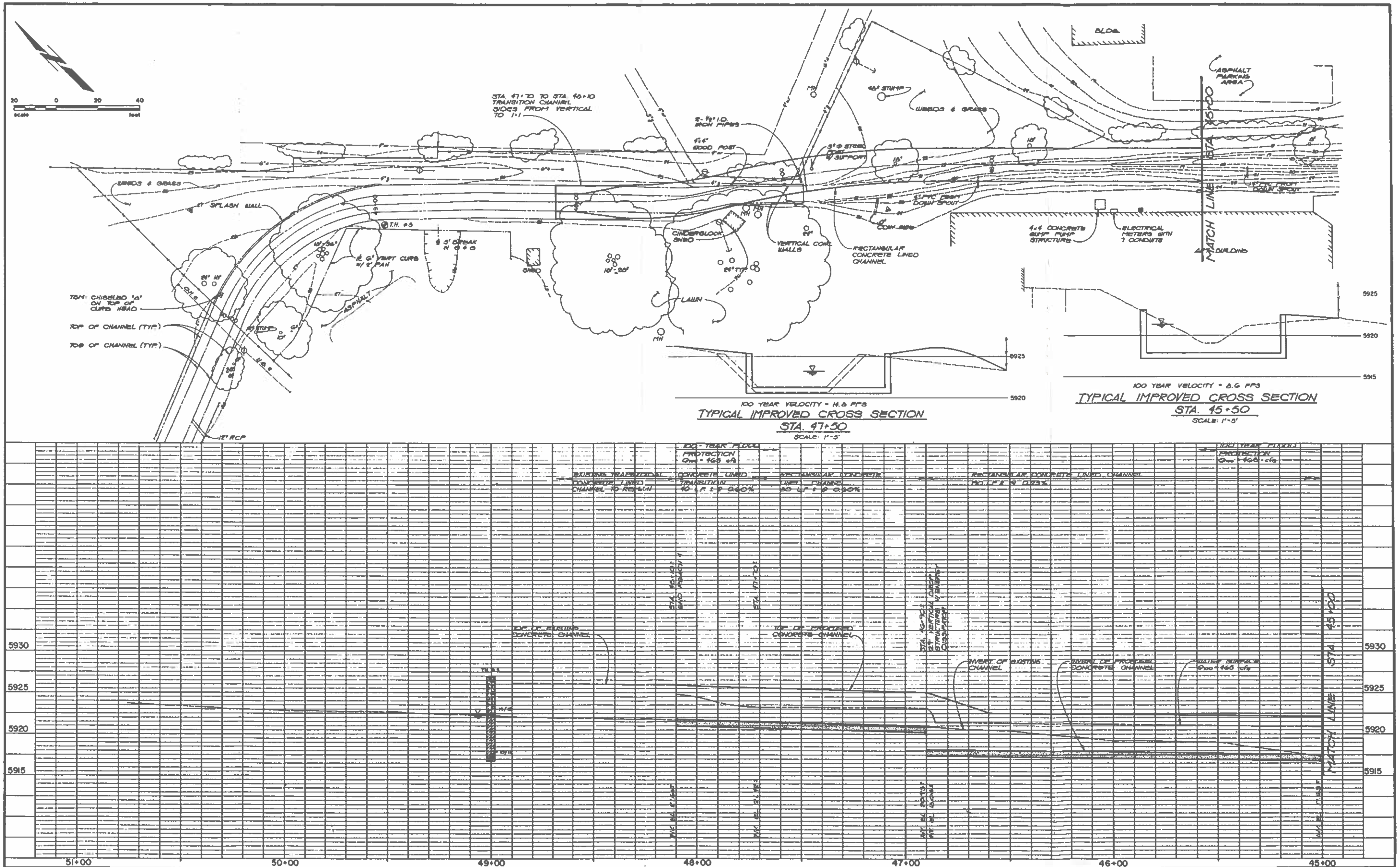
DESIGNED BY: TGF
DRAWN BY: JAF
CHECKED BY: CAM
DATE ISSUED: OCT. 1985

CITY OF COLORADO SPRINGS

PARK MEADOWS / CHEYENNE CREEK (SPRING RUN)
ALTERNATIVE 4-A
STA. 41+92 TO STA. 45+00

JOB NO.: 85433
SCALE: AS NOTED
SHEET NO.: 11 OF 16

COLORADO REGISTERED PROFESSIONAL ENGINEER 17194



NO.	REVISIONS	DATE	BY

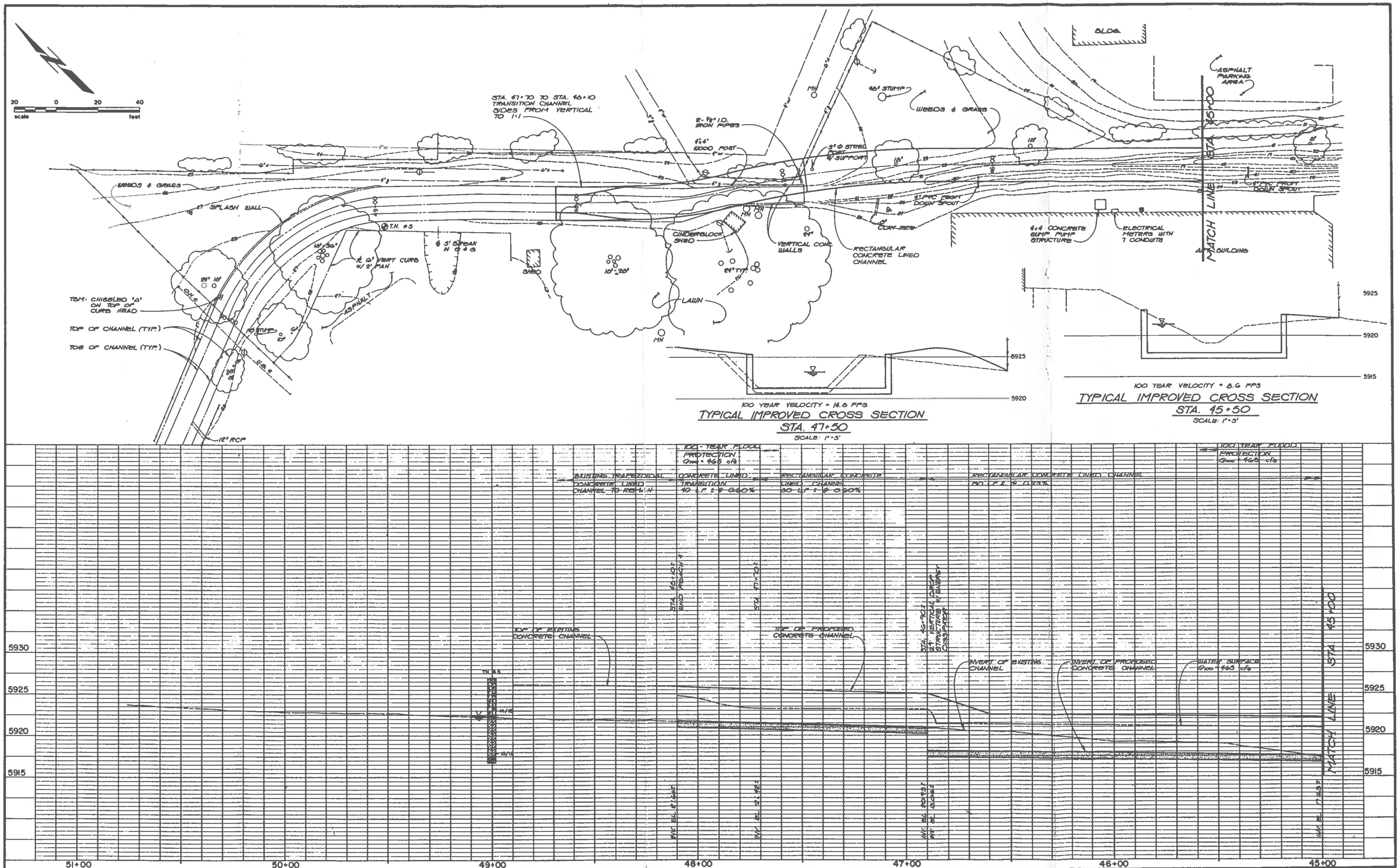
CONSULTING ENGINEERS
SELLARDS & GRIGG, Inc.
LAKEWOOD, COLORADO

TGF
JAF
CAM
OCT. 1985

CITY OF COLORADO SPRINGS

PARK MEADOWS / CHEYENNE CREEK
(SPRING RUN)
ALTERNATIVE 4-A
STA. 45+00 TO STA. 48+10

85433
AS NOTED
12 OF 16



NO.	REVISIONS	DATE	BY

COLORADO REGISTERED PROFESSIONAL ENGINEER 17191

CONSULTING ENGINEERS
SELLARDS & GRIGG, Inc.
LAKEWOOD, COLORADO

TGF	CITY OF COLORADO SPRINGS	85433
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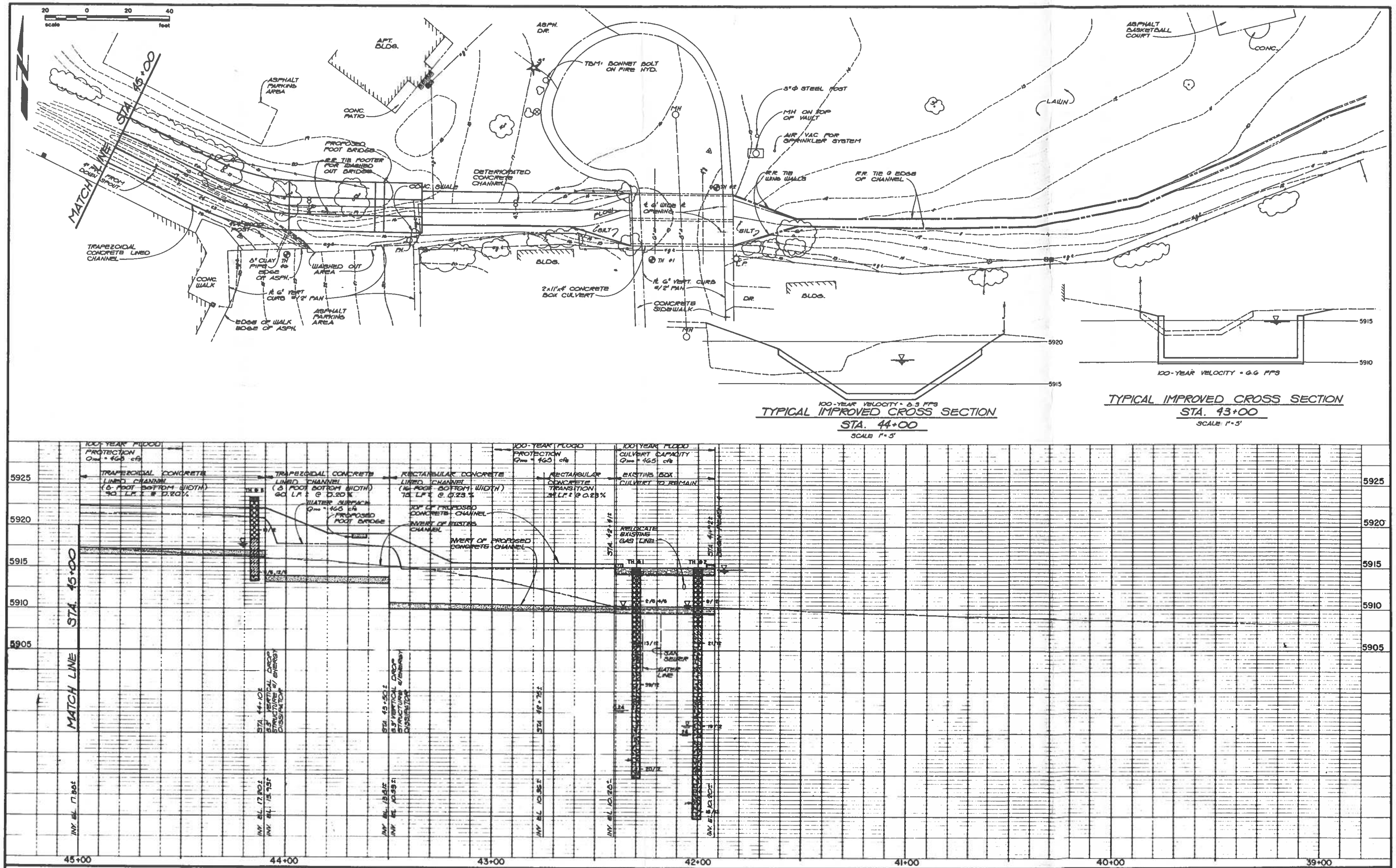
OCT. 1985		12 OF 16



NO.	REVISIONS	DATE	BY

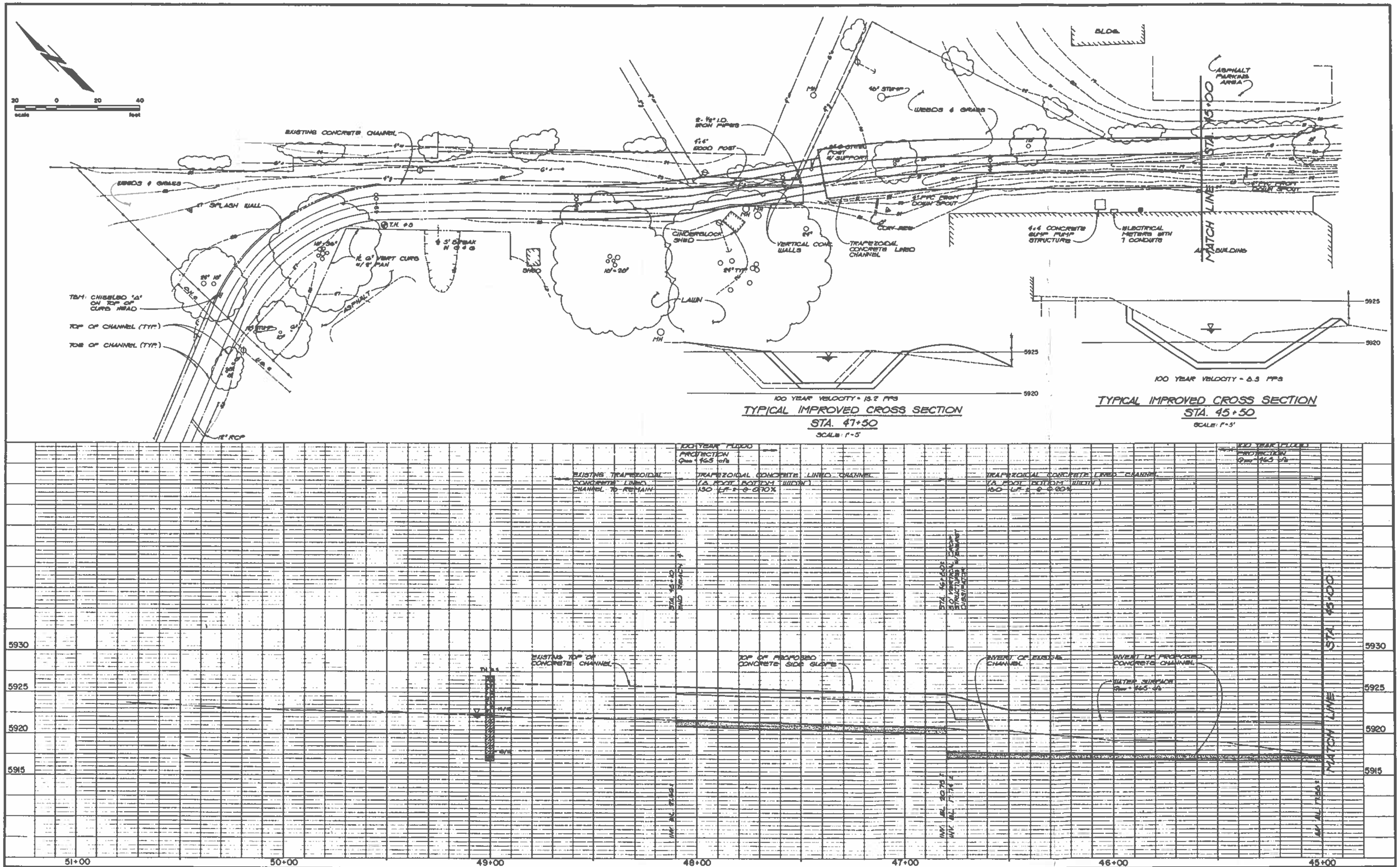
CONSULTING ENGINEERS
SELLARDS & GRIGG, Inc.
LAKEWOOD, COLORADO

DESIGNED BY TGF
DRAWN BY JAF
CHECKED BY CAM
DATE OCT. 1985

CITY OF COLORADO SPRINGS

PARK MEADOWS / CHEYENNE CREEK (SPRING RUN)
ALTERNATIVE 4-B
STA. 41+92 TO STA. 45+00

JOB NO. 85433
SCALE AS NOTED
SHEET 13 OF 16



NO.	REVISIONS	DATE	BY

CONSULTING ENGINEERS
SELLARDS & GRIGG, Inc.
LAKEWOOD, COLORADO

COLORADO REGISTERED PROFESSIONAL ENGINEER 17191

TGF	CITY OF COLORADO SPRINGS	85433
JAF	PARK MEADOWS / CHEYENNE CREEK (SPRING RUN) ALTERNATIVE 4-B STA. 45+00 TO STA. 48+10	
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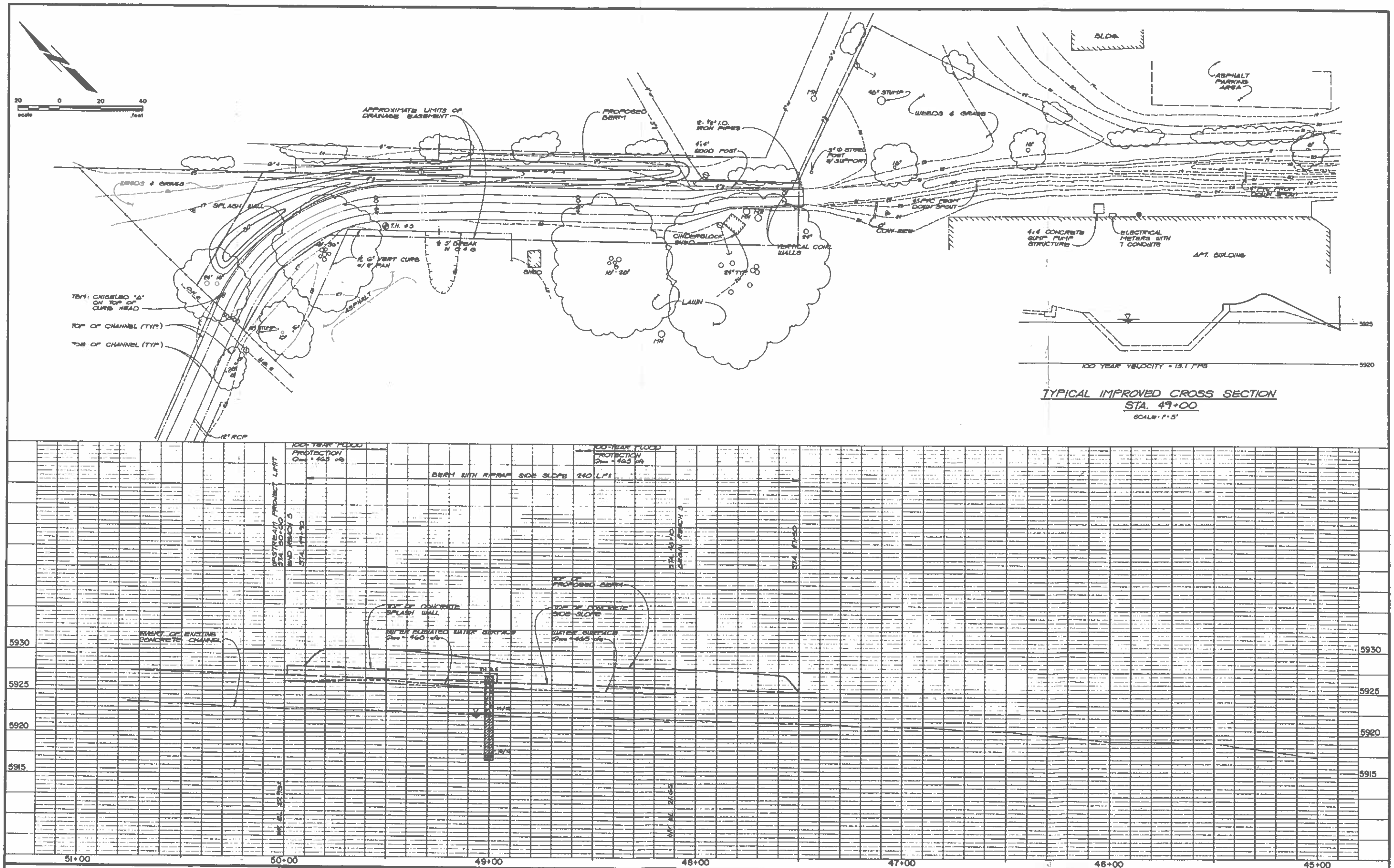
OCT. 1985		14 OF 16



NO.	REVISIONS	DATE	BY

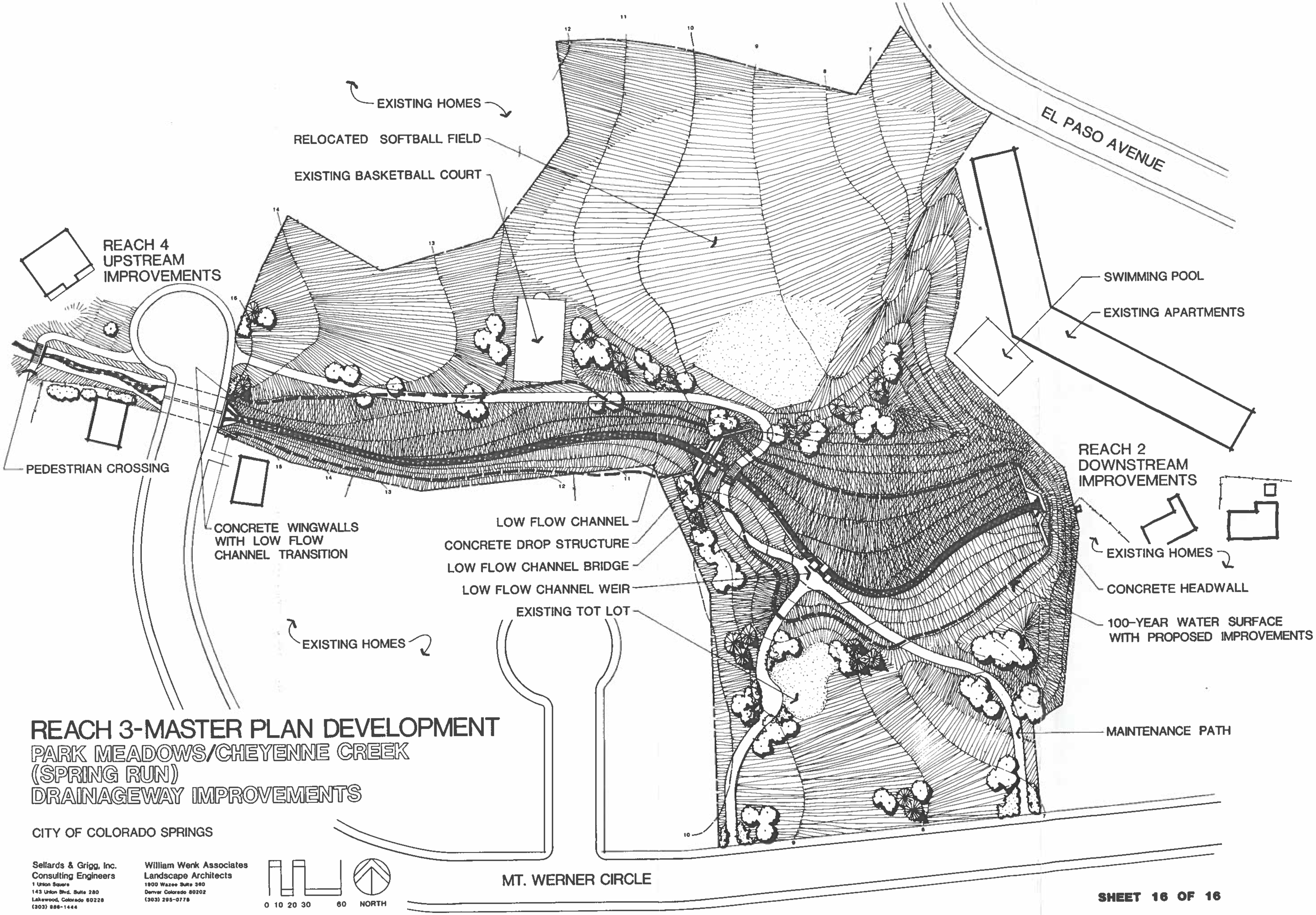
CONSULTING ENGINEERS
SELLARDS & GRIGG, Inc.
LAKEWOOD, COLORADO

TGF
JAF
CAM
OCT. 1985

CITY OF COLORADO SPRINGS

PARK MEADOWS / CHEYENNE CREEK
(SPRING RUN)
ALTERNATIVE 5-A
STA. 48+10 TO STA. 50+00

85433
AS NOTED
15 OF 16


EXISTING HOMES
RELOCATED SOFTBALL FIELD
EXISTING BASKETBALL COURT
EL PASO AVENUE
REACH 4
UPSTREAM
IMPROVEMENTS
SWIMMING POOL
EXISTING APARTMENTS
PEDESTRIAN CROSSING
REACH 2
DOWNSTREAM
IMPROVEMENTS
CONCRETE WINGWALLS
WITH LOW FLOW
CHANNEL TRANSITION
LOW FLOW CHANNEL
CONCRETE DROP STRUCTURE
LOW FLOW CHANNEL BRIDGE
LOW FLOW CHANNEL WEIR
EXISTING TOT LOT
EXISTING HOMES
CONCRETE HEADWALL
100-YEAR WATER SURFACE
WITH PROPOSED IMPROVEMENTS
EXISTING HOMES
MAINTENANCE PATH
REACH 3-MASTER PLAN DEVELOPMENT
PARK MEADOWS/CHEYENNE CREEK
(SPRING RUN)
DRAINAGEWAY IMPROVEMENTS
CITY OF COLORADO SPRINGS
Sellards & Grigg, Inc.
Consulting Engineers
1 Union Square
143 Union Blvd. Suite 280
Lakewood, Colorado 80228
(303) 986-1444
William Wenk Associates
Landscape Architects
1900 Wazee Suite 360
Denver Colorado 80202
(303) 295-0778
0 10 20 30 60
NORTH
MT. WERNER CIRCLE
SHEET 16 OF 16

APPENDIX B

Geotechnical Investigation

A. G. WASSENAAR, INC.
GEOTECHNICAL CONSULTANTS
COLORADO SPRINGS, COLORADO 80918
6755 EARL DRIVE, SUITE 105
303/590-9500

GEOTECHNICAL INVESTIGATION

PROPOSED CHANNEL IMPROVEMENTS
CHEYENNE CREEK/MEADOWS PARK AREA
COLORADO SPRINGS, COLORADO

PREPARED FOR

SELLARDS AND GRIGG, INC.
143 UNION BOULEVARD
SUITE 280
LAKEWOOD, CO 80228

PROJECT NO. 85059
AUGUST 26, 1985

A. G. WASSENAAR, INC.
GEOTECHNICAL CONSULTANTS
COLORADO SPRINGS, COLORADO 80918
6755 EARL DRIVE, SUITE 105
303/590-9500

August 26, 1985

SELLARDS AND GRIGG, INC.
143 Union Boulevard
Suite 280
Lakewood, CO 80228

ATTENTION: MR. CHUCK McKNIGHT

SUBJECT: GEOTECHNICAL INVESTIGATION
Proposed Channel Improvements
Cheyenne Creek/Meadows Park Area
Colorado Springs, Colorado
Project No. 85059

Gentlemen:

We have completed the geotechnical investigation for the proposed channel improvements at the subject site. Our summary of the data collected during our field and laboratory work and our analysis, opinions and conclusions are presented in the attached report. The purpose of our investigation is to provide design criteria for planning and site development, foundation systems, channel areas and other geotechnically related portions of the proposed improvements.

In general, the subsoils in the two proposed bridge areas consist of approximately 7.0 to 9.0 feet of man-made fill overlying relatively thin layers of medium stiff clay or loose sand. Medium dense to dense, gravelly sand is located at depths ranging from 9.5 to 10.0 feet. Competent bedrock was not encountered to the maximum depths explored of 25 and 30 feet. Ground water in the bridge areas ranges from 17.0 to 25.0 feet below the existing bridge roadway levels.

The subsoils along the existing channel alignment are more erratic and consist of zero to 5.0 feet of man-made fill overlying soft to stiff, sandy clay and very loose to medium dense, silty, clean to very clayey, gravelly sand. No bedrock was encountered to the depths explored of 4.0 to 10.5 feet. Ground water was measured at 3.5 feet below the ground surface in Test Hole No. 10. Ground water was not encountered in the other channel areas explored.

We recommend the bridge structures be founded on conventional box culvert construction or a spread footing-abutment type system bearing on the undisturbed, medium dense to dense sand subsoils beneath the existing man-made fill, clay and loose sand or on properly controlled, sand and gravel structural fill after the fill, clay and loose sand have been removed.

The subsoils along the creek should adequately support concrete-lined channel construction. Moderate swelling potential of the clay soils in one area was observed, however, we do not believe this condition should adversely affect the channel performance. It does not appear that ground water is flowing into the creek, therefore, we do not believe it is necessary to place drainage gravel beneath the channel.

Water soluble sulfate test results are not yet completed and will be sent under separate cover when ready.

Additional recommendations are presented in the following report.

If you have any questions, please do not hesitate to call our office. We have appreciated the opportunity to provide this service for you.

Sincerely,

A. G. WASSENAAR, INC.

Mark V. Herbert

MARK V. HERBERT, P. E.

COLORADO REGISTERED
MARK V. HERBERT
17895
PROFESSIONAL ENGINEER

MVH/dmw

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION
PROPOSED CHANNEL IMPROVEMENTS
CHEYENNE CREEK/MEADOWS PARK AREA
COLORADO SPRINGS, COLORADO

Purpose

This report presents results of a geotechnical investigation for the proposed channel improvements to be located in the Meadows Park area of Cheyenne Creek in Colorado Springs, Colorado. The site is located east of the intersection of Southgate Road and Rice Drive. The investigation was made to assist in determining design criteria for preliminary planning and site development, the type, depth, and design criteria for the foundation systems of bridges, slab-on-grade channel areas and any geotechnically related special precautions which should be taken. Factual data gathered during the field and laboratory work is summarized on Figures 1 through 11 attached. Our opinions and recommendations presented in this report are based on the data obtained during the field investigation, laboratory testing, and our experience with similar type projects.

Proposed Construction

The proposed channel improvements will consist of two possible bridge replacements along Mt. Werner Circle, and providing a concrete-lined channel or equivalent structure to increase the flow capacity of Cheyenne Creek through the project area.

The bridges, we assume, will consist of concrete and steel construction, and have been referred to as Mt. Werner Circle West and East, respectively, in this report. The project commences at a location behind the residence at 2027 South Corona Avenue and terminates behind the residence at 1177 Mt. Werner Terrace. The length of Cheyenne Creek through the project area is approximately 3,000 feet. The locations of our test holes are shown on Figure 1 and the existing creek elevations on Figures 2, 3 and 4. We should be notified to review this report when channel and bridge elevations have been defined.

Site Conditions

The westerly 250 feet and the easterly 500 feet of Cheyenne Creek currently have concrete channels. Both portions are in good condition, however, the easterly portion is partially clogged with sediment. Other areas consist of an earth channel or a badly cracked concrete channel. The bridge structures consist of two cell box culvert type construction and appear to be in reasonable condition. Cheyenne Creek flows in an easterly direction through the project area and appeared to be at low flow during this investigation. The elevation difference between the upstream and downstream extremities was not known at the time of this investigation. Vegetation along the creek consists of various grasses, weeds and trees. No bedrock outcrops were observed on the site.

Investigations

Subsurface conditions were investigated by drilling eleven test borings at the locations indicated on Figure 1. Eight of the borings were advanced using a 4-inch diameter, continuous flight auger powered by a CME 45 drilling rig. At frequent intervals, samples of the subsoils were taken using a California sampler which is driven into the soil by dropping a 140-pound hammer through a free fall of 30 inches. The California sampler is a 2.5-inch outside diameter by 2-inch inside diameter device. The number of blows required to drive the sampler into the soils is known as a penetration test. The number of blows required for the sampler to penetrate 12 inches gives an indication of the consistency or relative density of the soils encountered. Results of the penetration tests are presented on the Logs of Exploratory Borings, Figures 2 and 3.

Test Hole Nos. 9, 10 and 11 were excavated manually. Denver penetrometer tests were taken at shallow elevations. The results of these tests are presented on the Logs of Exploratory Borings, Figure 4.

Laboratory Testing

Samples were returned to the laboratory where they were visually classified and appropriate testing assigned to specific samples to evaluate their engineering properties. The laboratory tests included five settlement-swell tests to evaluate the effect of wetting and loading on the soils. The results of the settlement-swell tests are presented on Figures 5, 6 and 7. Ten gradation analysis tests and six Atterberg limits tests were conducted to evaluate grain size distribution and plasticity of selected samples. These results are presented on Figures 8, 9, 10 and 11. In addition, hydrometer tests were conducted to determine the clay-sized fraction of selected samples and water soluble sulfate tests were performed to determine the detrimental effect of the subsoils on concrete.

Subsoil Conditions

Bridge Structure, Mt. Werner Circle West, Test Hole Nos. 1 and 2

Our test holes indicate the subsoils, in general, consist of approximately 7.0 to 9.0 feet of man-made, sandy clay fill with occasional gravel. A two-foot layer of medium stiff, sandy clay is located at 7.0 to 9.0 feet below the ground surface in Test Hole No. 2. Medium dense to dense gravelly, clean to slightly clayey sand with cobbles is located at 9.0 feet below the ground surface and extended to the maximum depths explored of 25 and 30 feet. No competent bedrock was encountered. Ground water was measured at depths of 17.0 to 19.0 feet at the time of drilling and at depths of 17.0 to 20.0 feet 24 hours after drilling. The test holes caved at depths of 23.0 to 28.0 feet. A more complete description of the subsoils and ground water is shown on Figure 2.

Based upon our field and laboratory investigation, the man-made clay fill exhibited in-situ densities ranging from 95.4 to 103.7 pounds per cubic foot, and in-situ moistures ranging from 19.8 to 23.9 percent. These soils also exhibited moderate potential for consolidation. The man-made fill should not be used to support the bridge. The underlying medium dense to dense sand is non-expansive and should exhibit low potential for consolidation under anticipated bridge loading.

Bridge Structure, Mt. Werner Circle, East, Test Hole Nos. 3 and 4

Our test holes indicate the subsoils at this bridge site, in general, consist of approximately 8.0 feet of man-made sandy clay or gravelly sand fill overlying a thin layer of very loose to loose, silty sand. Medium dense to dense, gravelly sand with cobbles is located at a depth of 10.0 feet and extended to depths of 25.0 and 29.0 feet. Weathered claystone bedrock was encountered at a depth of 29.0 feet in Test Hole No. 4. No competent bedrock was encountered. Ground water was measured at depths of 24.5 to 27.0 feet at the time of drilling and at a depth of 25.0 feet 24 hours after drilling in Test Hole No. 4. The test holes caved at depths of 19.0 to 27.0 feet which was above the water table in Test Hole No. 3. A more complete description of the subsoils and ground water is shown on Figure 2.

The man-made clay fill at this bridge site exhibited an in-situ density of 95.3 pounds per cubic foot, and an in-situ moisture content of 12.6 percent. These fill soils exhibited high consolidation potential and should not be used to support the bridge structure. The underlying loose sand exhibits moderate consolidation potential, while the medium dense to dense, gravelly sand should exhibit low potential for consolidation under anticipated bridge loadings.

Channel Areas, Test Hole Nos. 5 through 11

Our test holes indicate the subsoils, in general, consist of approximately zero to 5.0 feet of interbedded man-made, sand and clay fill overlying soft to stiff, silty, sandy, clay with occasional gravel; medium dense clayey sand and very loose to medium dense, silty, gravelly sand. The soft clay was observed in Test Hole Nos. 8, 9 and 10. No competent bedrock was encountered to the depths explored of 4.0 to 10.5 feet. Ground water was measured at a depth of 3.5 feet 72 hours after excavating in Test Hole No. 10. Ground water was not observed in the remaining test holes. Test Hole Nos. 6, 7 and 8 caved at depths of 7.0 to 9.0 feet. A more complete description of the subsoils and ground water is shown on Figures 3 and 4.

Based upon our field and laboratory investigations, the clay soils along the channel exhibited in-situ densities ranging from 95.7 to 111.4 pounds per cubic foot, and in-situ moistures ranging from 13.8 to 24.2 percent. The stiff clay soils exhibited moderate swell potential (see Figure 6) while the soft to medium stiff clay exhibits moderate to high potential for consolidation (see Figure 7). Discussion of the channel areas is provided later in the report.

Foundation Recommendations

A suitable foundation system for the proposed bridge structures would be conventional box culvert type construction or a spread footing/abutment type system bearing on the natural, undisturbed medium dense to dense gravelly sand soils beneath the existing fill, clay and loose sand, or on properly controlled, non-expansive, sand and gravel structural fill after the fill, clay and loose sand have been removed. The footings or box culverts should be designed for a soil pressure not to exceed 3,000 pounds per square foot based upon dead load plus live load. The live load should include HS-20 truck loading. The following design criteria should also be observed:

a) Using the allowable soil pressure recommended above, we estimate the maximum settlement for the bridges will be in the order of 3/4-inch with differential settlement of less than 1/2-inch. Footings should be proportioned as much as practicable to minimize differential movement.

b) Continuous concrete foundation walls should be designed to span a localized settlement of 10 feet.

c) The base for structural fill should include all areas within a 1:1 horizontal to vertical slope from the edge of the footings or culverts.

d) Prior to placement of fill or concrete, the excavations should be inspected by a Soils Engineer to insure that all existing man-made fill materials, soft or medium stiff clay and loose sand have been removed.

e) Structural fill under footings or culverts should be placed in six-inch maximum, loose lifts at optimum moisture content and compacted to 100 percent of standard Proctor density according to ASTM D698.

f) Proposed off-site material to be used for structural fill should be approved by a Soils Engineer prior to hauling to the site. Structural fill should be closely controlled by a Soils Engineer. A guide specification regarding the quality of material and placement and compaction procedures is attached. Fill material should generally consist of a well-graded, pit-run sand and gravel.

We also considered a driven steel H-pile as the foundation system for these bridges, however, cobbles are scattered in the medium dense to dense sand and gravel. The cobbles range up to 6 to 8 inches in size and may damage or deflect the piles during installation. Also, the bridge sites are in residential areas and pile driving may cause vibrations and minor damage to the residences.

Lateral Loads

We assume there will be horizontal loads into the top of foundation elements of the proposed bridges. These loads will be caused by earthquake motions, wind loads and hydraulic forces. These loads should be resisted by frictional forces along the footing-soil interface and by passive earth pressure.

We recommend a coefficient of sliding friction of 0.50 be used for footings on the undisturbed gravelly sand or properly compacted structural fill. Keyways cut into the bearing soils should be designed for a passive earth resistance of 320 pounds per cubic foot, equivalent fluid pressure. If passive earth resistance is required against abutment or box culvert walls, a value of 160 pounds per cubic foot, equivalent fluid pressure, should be used. This assumes backfill behind the walls will consist of the on-site sandy clay soil.

Lateral Earth Pressures

Lateral pressures on abutment or box culvert walls depend upon the type of wall, hydrostatic pressure behind the wall, type of backfill material, and allowable wall movements. We recommend hydrostatic pressures be minimized by placing a perimeter drain system at the wall base. Where anticipated wall movements are less than approximately 0.5 percent of the wall height or wall movement is constrained, lateral earth pressures should be estimated for an "at rest" condition. We believe the "at rest" condition should be used for these bridges. Walls backfilled with sandy clay material should be designed for an equivalent fluid lateral earth pressure of 60 pcf for the "at rest" condition. If walls are backfilled with a free-draining granular backfill such as a clean sand or gravel, an equivalent fluid lateral earth pressure of 50 pcf for the "at rest" condition should be assumed.

Channel Areas

In our opinion, the majority of the soils along the exisiting creek alignment will adequately support a concrete channel, or slab-on-grade type construction. The soft to medium stiff clay in the areas of Test Hole Nos. 8, 9 and 10 should be evaluated, prior to placing concrete, to determine their suitability for slab-on-grade construction. Over-excavation of unsuitable clay soils and replacement with a better quality clayey sand material may be required.

The clay soils in the general area of Test Hole No. 5 exhibit moderate swelling potential. The existing concrete channel in this area appeared to be in good condition and showed no signs of distress caused by swelling soils. Although some damage from swelling soils should be anticipated, we do not believe the performance of the channel will be adversely affected.

We recommend the sides of the channel be no steeper than, 1 to 1, horizontal to vertical, to minimize lateral earth pressures against the channel walls. We suggest heavy wire mesh reinforcement or moderately spaced reinforcing bars to reduce differential movement of the slabs.

Based on our exploratory borings, ground water measurements and existing flow elevations, it does not appear that ground water is contributing to the flow in the creek. In our opinion, in most areas, the creek water is "perched" in the overlying clay and clayey sand strata and is not continuous with the underlying ground water table. Therefore, we do not believe that a layer of drainage gravel or fabric beneath the channel is necessary. We also recommend eliminating weep holes in most areas. It may be desirable to install weep holes in the general area of Test Hole No. 10, since shallow ground water was observed at this location.

Structural Fill Soils

We estimate up to four feet of structural fill will be necessary under bridge foundations. Where fill soils are necessary, a suitable off-site soil should be used for structural fill beneath bridge footings and culverts. The soils should be placed in six-inch maximum, loose lifts at optimum moisture content and compacted to at least 100 percent of standard Proctor density, according to ASTM D698. All existing man-made fill, clay and loose sand soils should be removed prior to placement of structural fill beneath bridge footings. Backfill against walls should be compacted to 95 percent of ASTM D698. Off-site material considered for structural fill should be tested and approved by a Soils Engineer prior to hauling to the site. Attached is a guide specification for placement and compaction of structural fill.

Construction Excavations

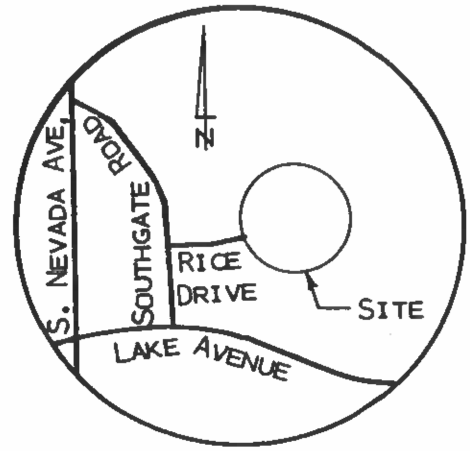
As mentioned previously, ground water should not present a problem during construction. We suggest that creek water be diverted away from or around construction areas by using pipes, conduits and pumps. We anticipate that proper water diversion will maintain reasonably dry working conditions. Proper diversion of creek water will also reduce the possibility of unstable bearing soils being caused by excess moisture. If unstable soils are encountered, they should be over-excavated to firm soils and replaced with compacted pit-run sand and gravel fill.

Limitations

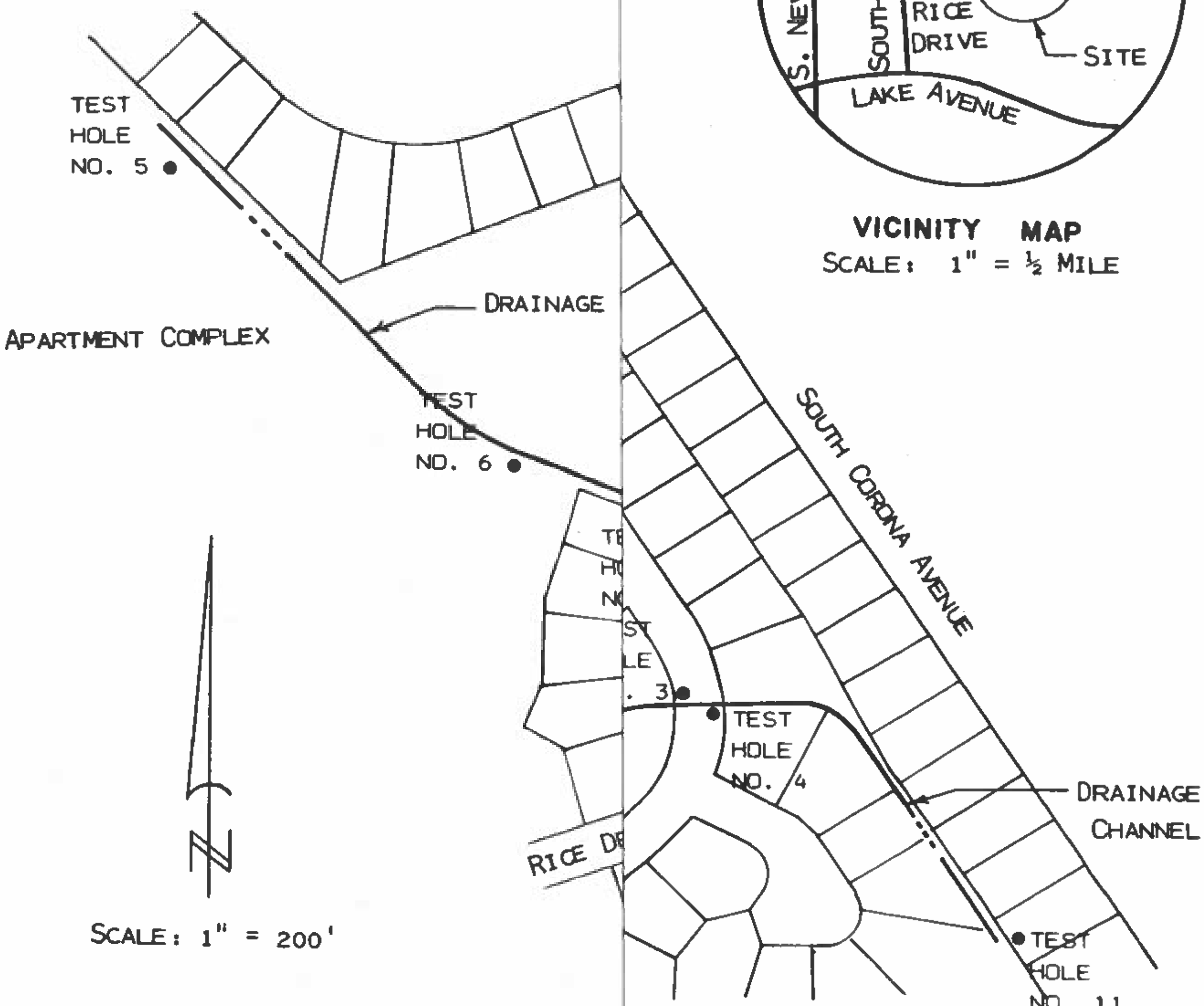
The test holes drilled for this study were spaced to obtain a reasonably accurate picture of subsurface conditions for design purposes. Variations in subsoil conditions not indicated by the borings are possible. If unexpected subsoil conditions are observed during construction, we should be called to review our recommendations. The professional judgments expressed in this report meet the standard care of our profession. The completed excavations and placement and compaction of fill should also be inspected by a Soils Engineer.

If you have any questions concerning this report or the investigation, do not hesitate to contact our office.

MVH/dmw

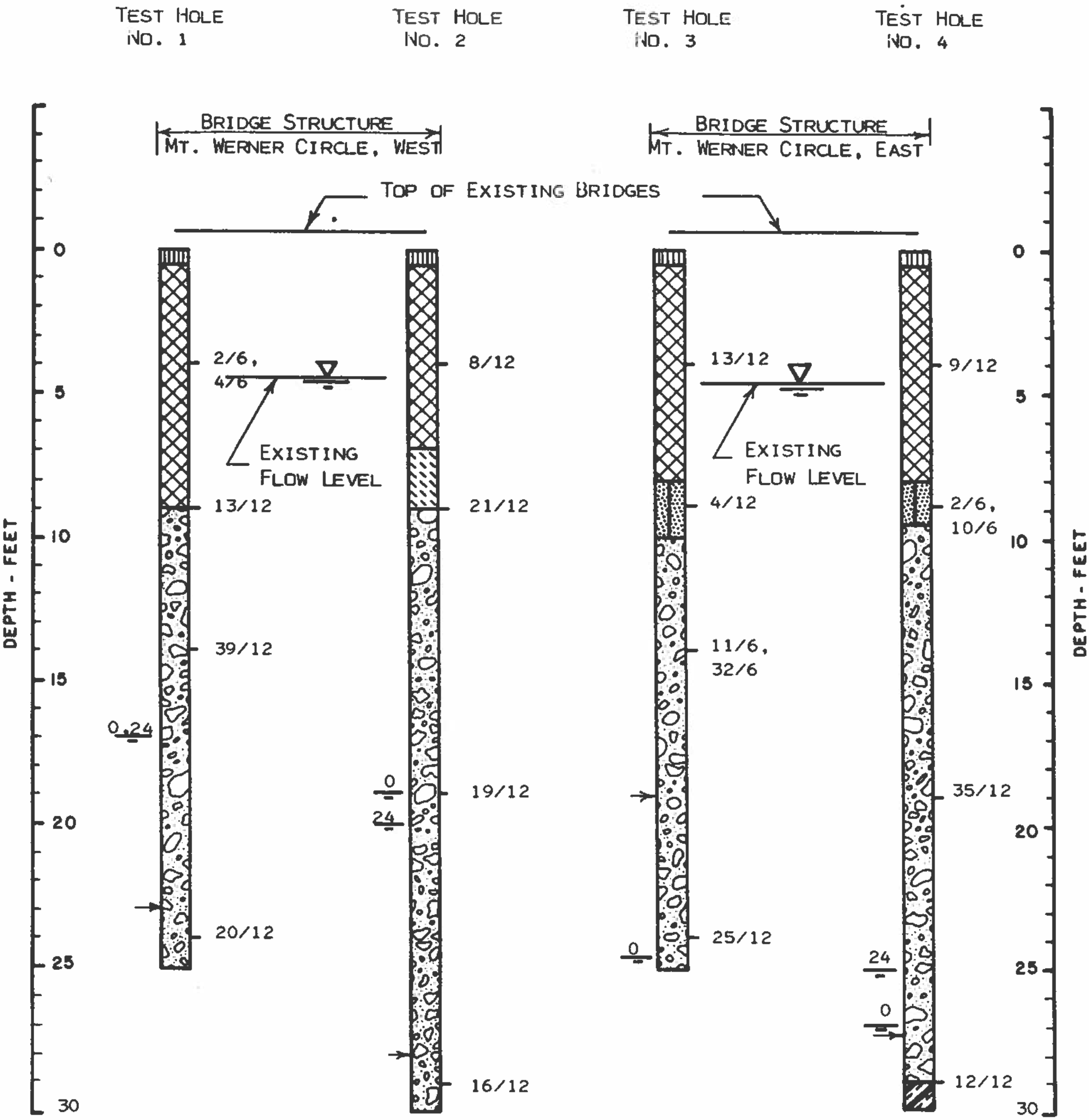


VICINITY MAP
SCALE: 1" = ½ MILE



SCALE: 1" = 200'

A.G. WASSENAAR, INC. **GEOTECHNICAL CONSULTANTS**	
CHANNEL IMPROVEMENTS LOCATION OF TEST HOLES SITE PLAN	FIGURE ONE PROJECT NO. 85059 DATE AUG. 23, 1985



LEGEND

ASPHALT AND BASE COURSE

FILL, MAN-MADE, CLAY AND SAND, SILTY, OCCASIONAL GRAVEL, MOIST TO VERY MOIST, REDDISH BROWN TO DARK BROWN (AF)

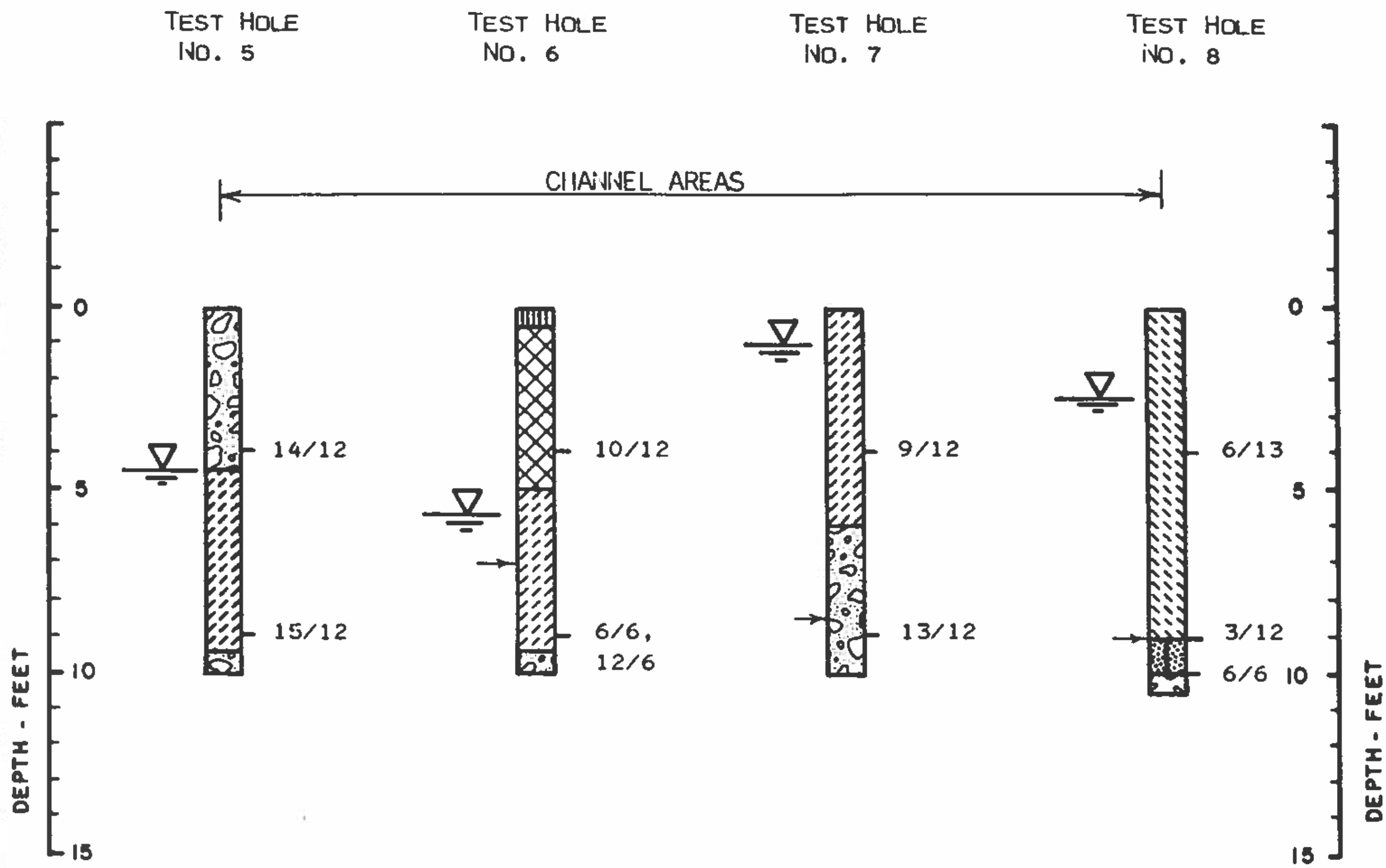
CLAY, SOFT TO MEDIUM STIFF, SILTY TO VERY SILTY, SANDY WITH CLAYEY SAND LENSES, MOIST TO VERY MOIST, GOLD BROWN TO DARK BROWN (CL)

SAND, VERY LOOSE TO LOOSE, SILTY, MOIST, BROWN TO GOLD BROWN (SM)

SAND, MEDIUM DENSE TO DENSE, CLEAN TO SLIGHTLY CLAYEY, CLEAN TO SILTY, OCCASIONAL GRAVEL TO VERY GRAVELLY WITH THICK COBBLE LAYERS, MOIST TO WET, LIGHT BROWN TO REDDISH BROWN (SP, SM, SW)

NOTES (SEE FIGURE 4)

LOGS OF EXPLORATORY BORINGS



LEGEND

ASPHALT AND BASE COURSE

FILL, MAN-MADE, CLAY AND SAND INTERBEDDED, SILTY, OCCASIONAL GRAVEL, MOIST TO VERY MOIST, BROWN TO DARK BROWN (AF)

CLAY, MEDIUM STIFF, SILTY, SANDY, VERY MOIST, DARK BROWN WITH GOLD BROWN LENSES (CL)

CLAY, STIFF, SILTY TO VERY SILTY, SANDY, OCCASIONAL GRAVEL, MOIST TO VERY MOIST, BROWN TO DARK BROWN (CL)

SAND, VERY LOOSE TO LOOSE, SILTY, OCCASIONAL GRAVEL, MOIST TO WET, REDDISH BROWN (SP, SM)

SAND, MEDIUM DENSE, CLEAN TO SILTY, CLEAN TO SLIGHTLY CLAYEY, GRAVELLY, MOIST TO WET, REDDISH BROWN (SP, SM, SW)

CLAY (WEATHERED CLAYSTONE), MOIST, GOLD BROWN

14/12 INDICATES THAT 14 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES ARE REQUIRED TO DRIVE A 2-INCH DIAMETER SAMPLER 12 INCHES.

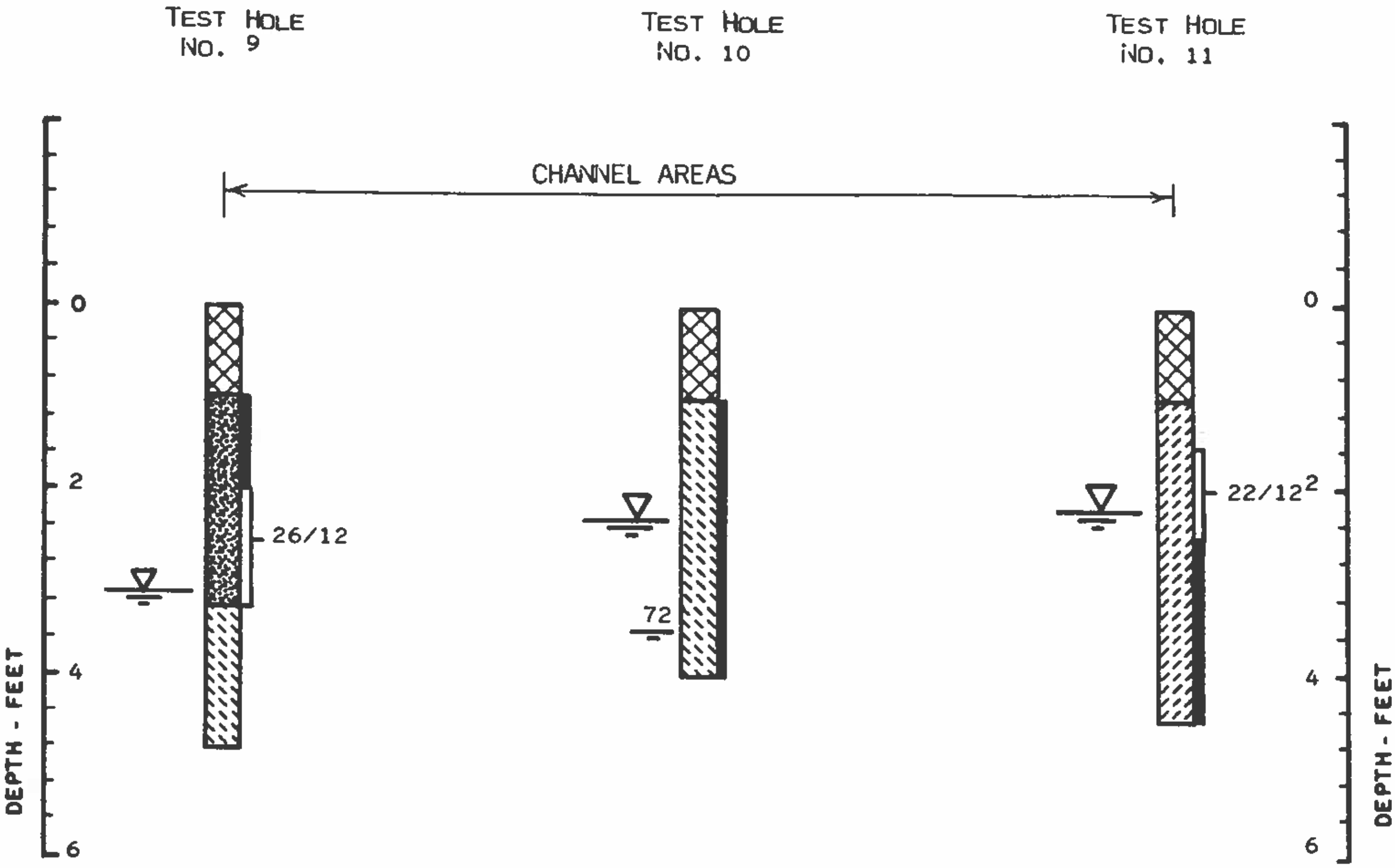
0 INDICATES THE DEPTH TO THE FREE WATER TABLE AND THE NUMBER OF HOURS AFTER DRILLING THE MEASUREMENT WAS TAKEN.

INDICATES THE DEPTH AT WHICH THE TEST BORING CAVED.

INDICATES WATER ELEVATION IN CREEK.

NOTES (SEE FIGURE 4)

LOGS OF EXPLORATORY BORINGS



LEGEND

FILL, MAN-MADE, SAND AND CLAY, SILTY, OCCASIONAL DEBRIS, MOIST TO VERY MOIST, REDDISH BROWN TO DARK BROWN (AF)

SAND, MEDIUM DENSE, SILTY, CLEAN TO VERY CLAYEY, MOIST TO VERY MOIST, REDDISH BROWN TO DARK BROWN (SC)

CLAY, SOFT TO MEDIUM STIFF, SILTY BROWN (CL)

CLAY, STIFF, SILTY, SANDY TO VERY SANDY, MOIST, DARK BROWN (CL)

26/12 INDICATES THAT 26 BLOWS OF A 15-POUND HAMMER FALLING 18 INCHES ARE REQUIRED TO DRIVE A 1-INCH DIAMETER ROD 12 INCHES INTO THE SUBSOILS.

INDICATES DEPTH AT WHICH AN AUGER, BUCKET, OR BAG SAMPLE WAS OBTAINED.

72 INDICATES THE DEPTH TO THE FREE WATER TABLE AND THE NUMBER OF HOURS AFTER DRILLING THE MEASUREMENT WAS TAKEN.

INDICATES WATER ELEVATION IN CREEK.

NOTES

1. TEST HOLES WERE DRILLED AUGUST 16, 1985 WITH A 4-INCH DIAMETER CONTINUOUS FLIGHT POWER AUGER.
2. TEST HOLE NOS. 9, 10 AND 11 WERE EXCAVATED MANUALLY ON AUGUST 14, 1985.
3. ELEVATIONS OF THE TEST HOLES AND TEST PITS WERE NOT KNOWN AT THE TIME OF THIS INVESTIGATION.
4. DRILL LOGS SHOWN IN THIS REPORT ARE SUBJECT TO THE LIMITATIONS, EXPLANATIONS, AND CONCLUSIONS OF THE REPORT.

LOGS OF EXPLORATORY BORINGS

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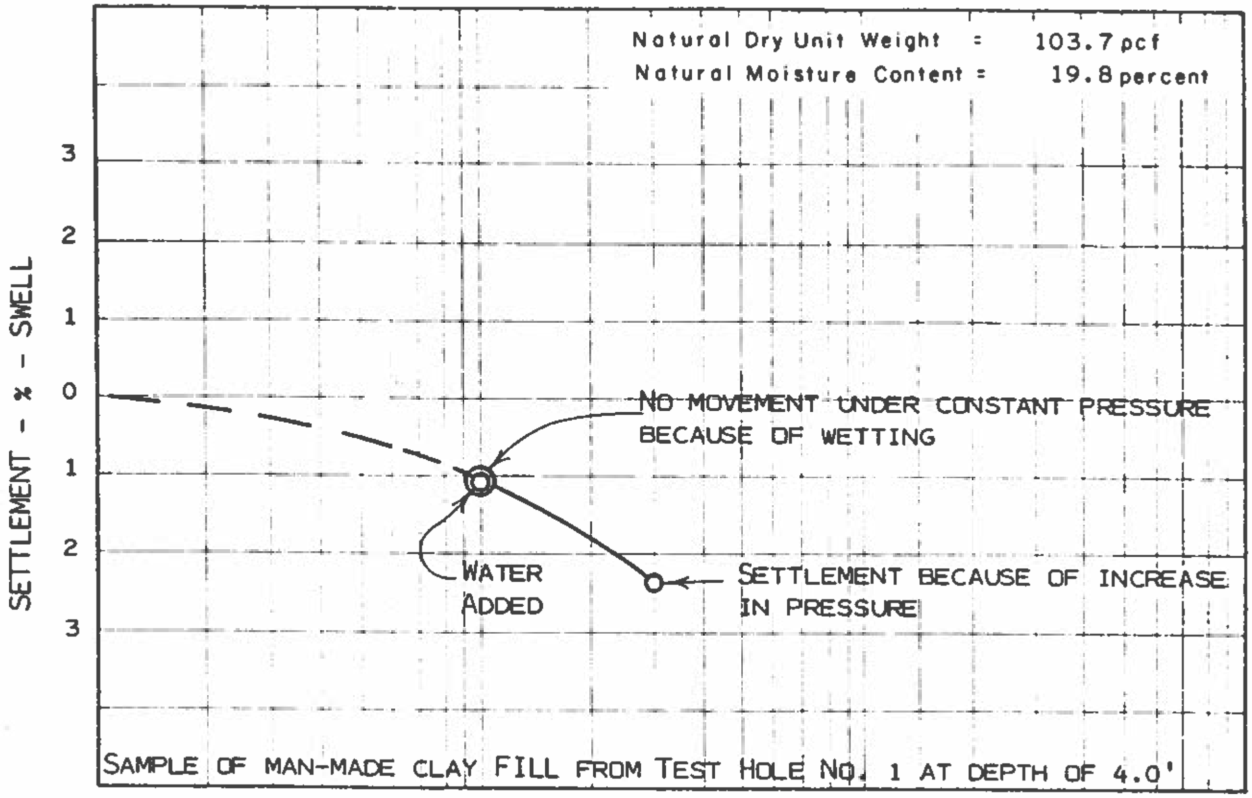

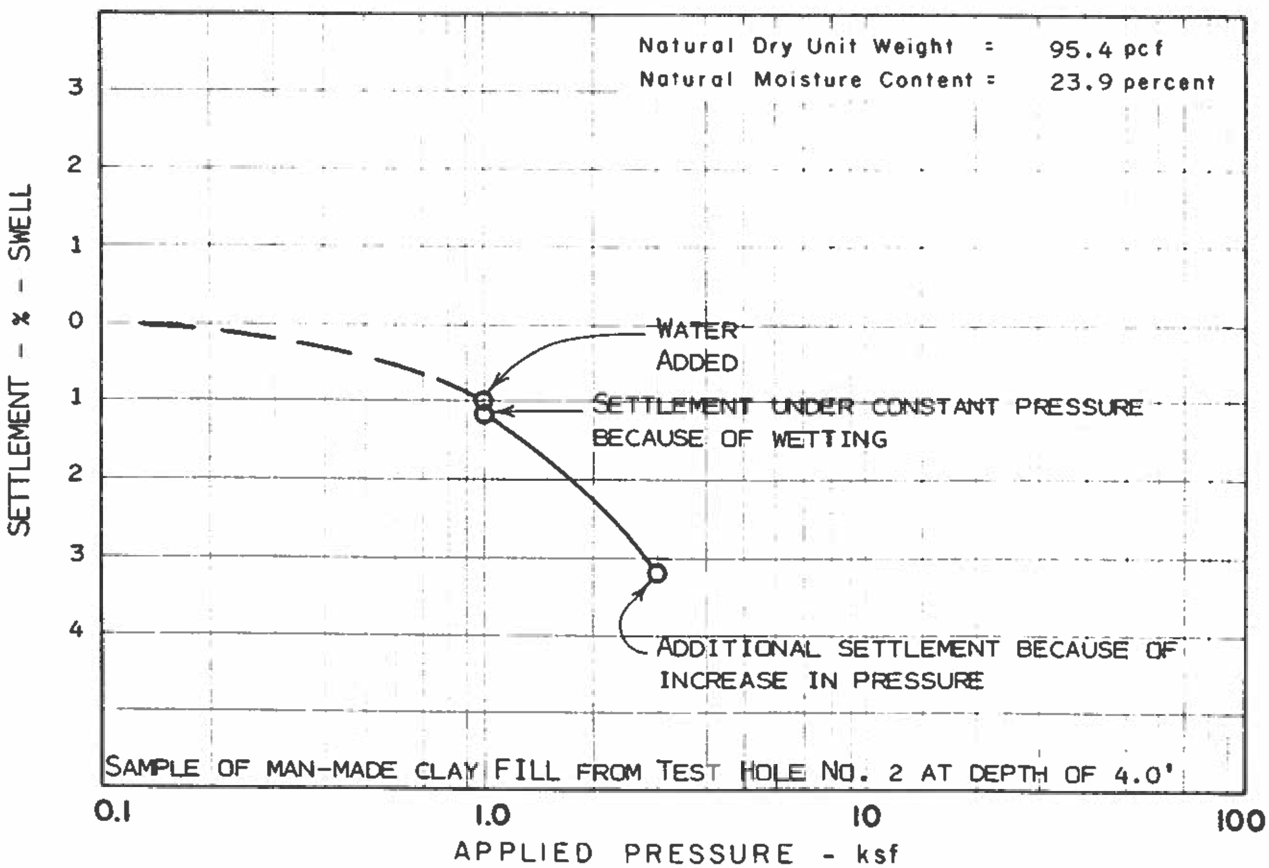

Settlement — Swell Test Results
FIGURE 5

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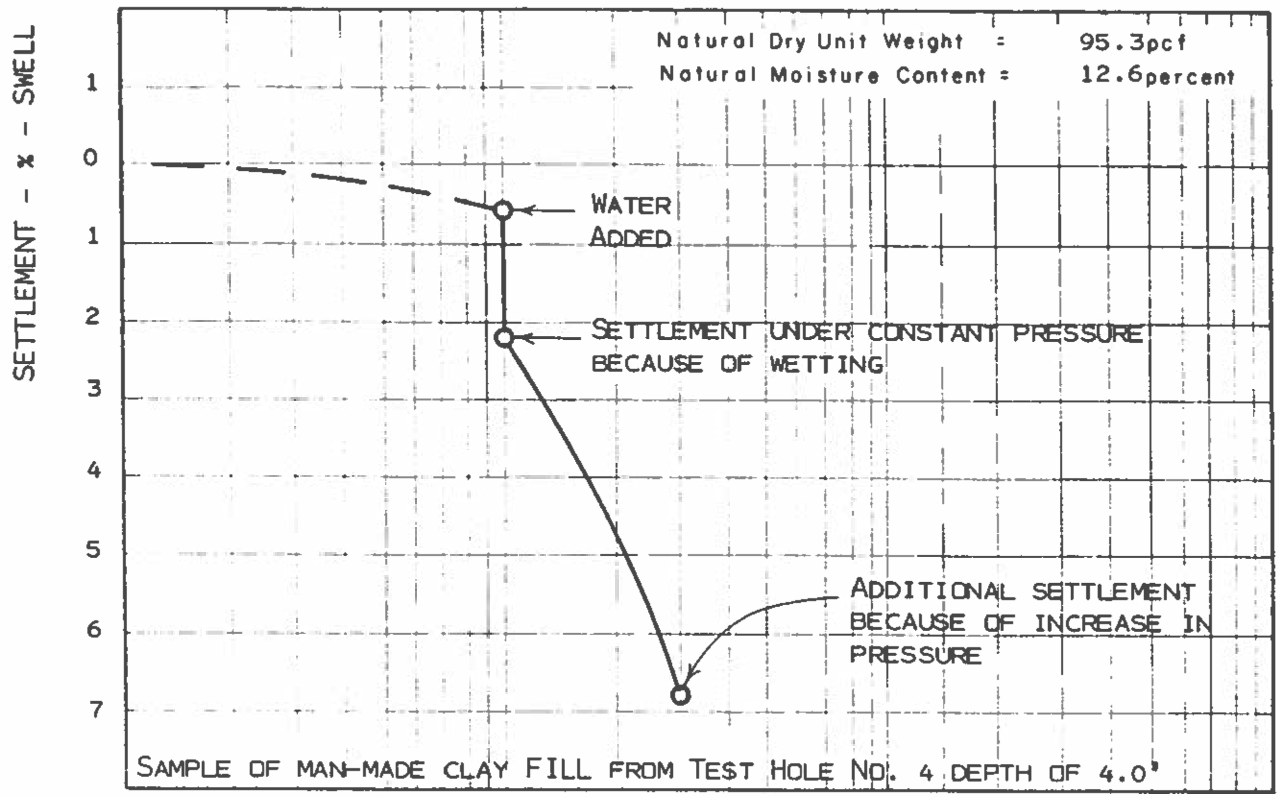

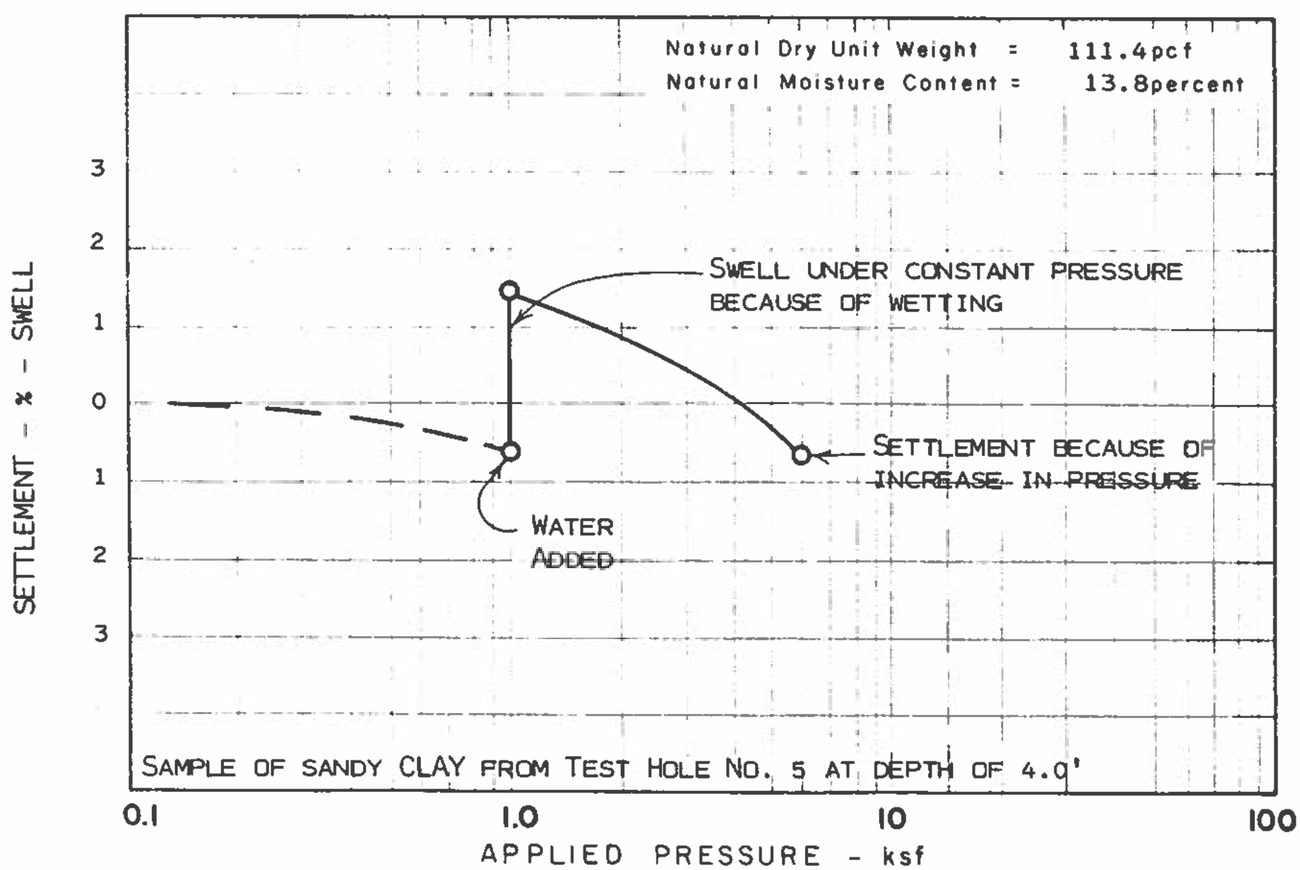

Settlement — Swell Test Results
FIGURE 6

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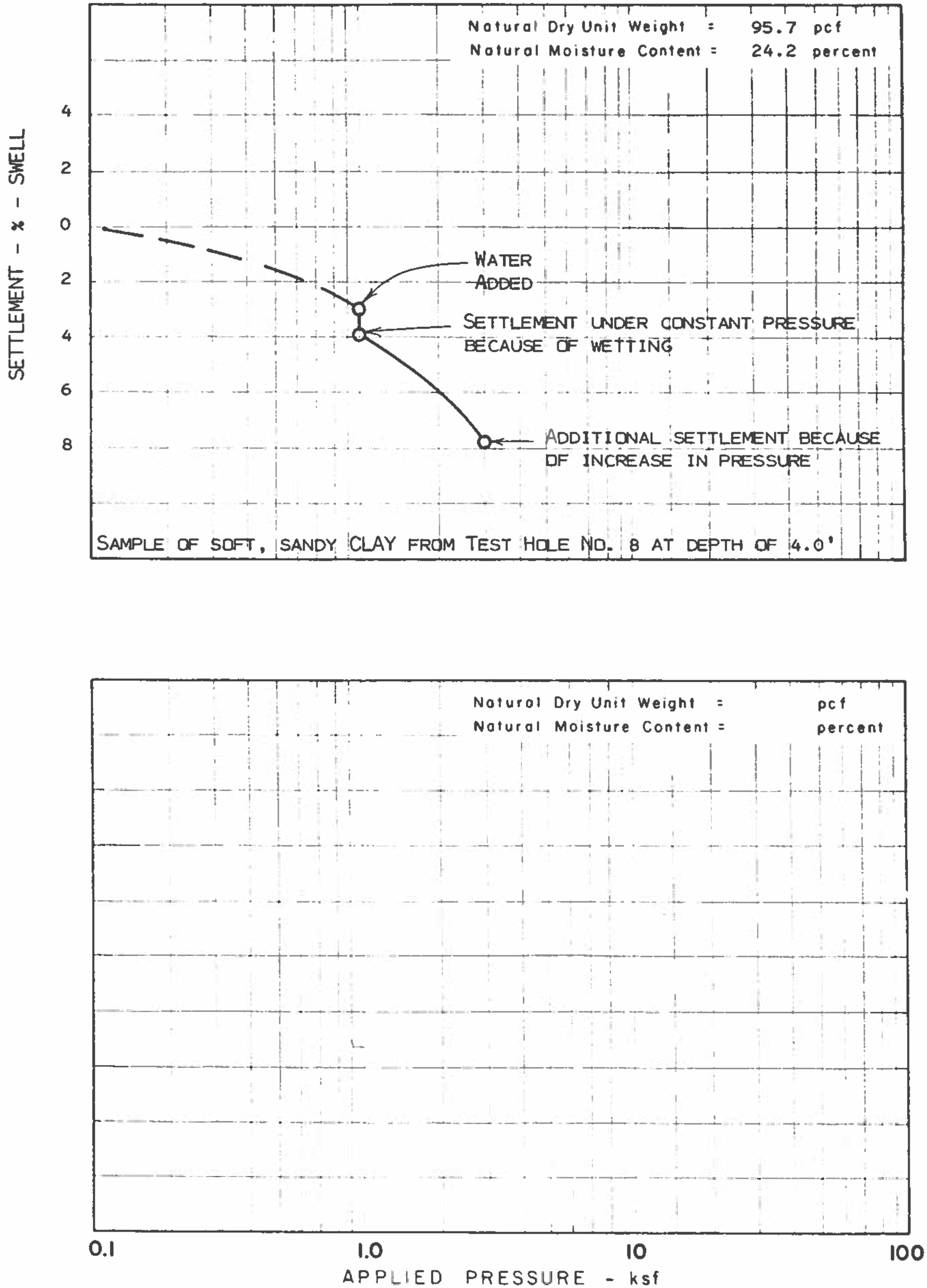

Settlement — Swell Test Results

A. G. WASSENAAR, INC.
GEOTECHNICAL CONSULTANTS

GRADATION TEST RESULTS

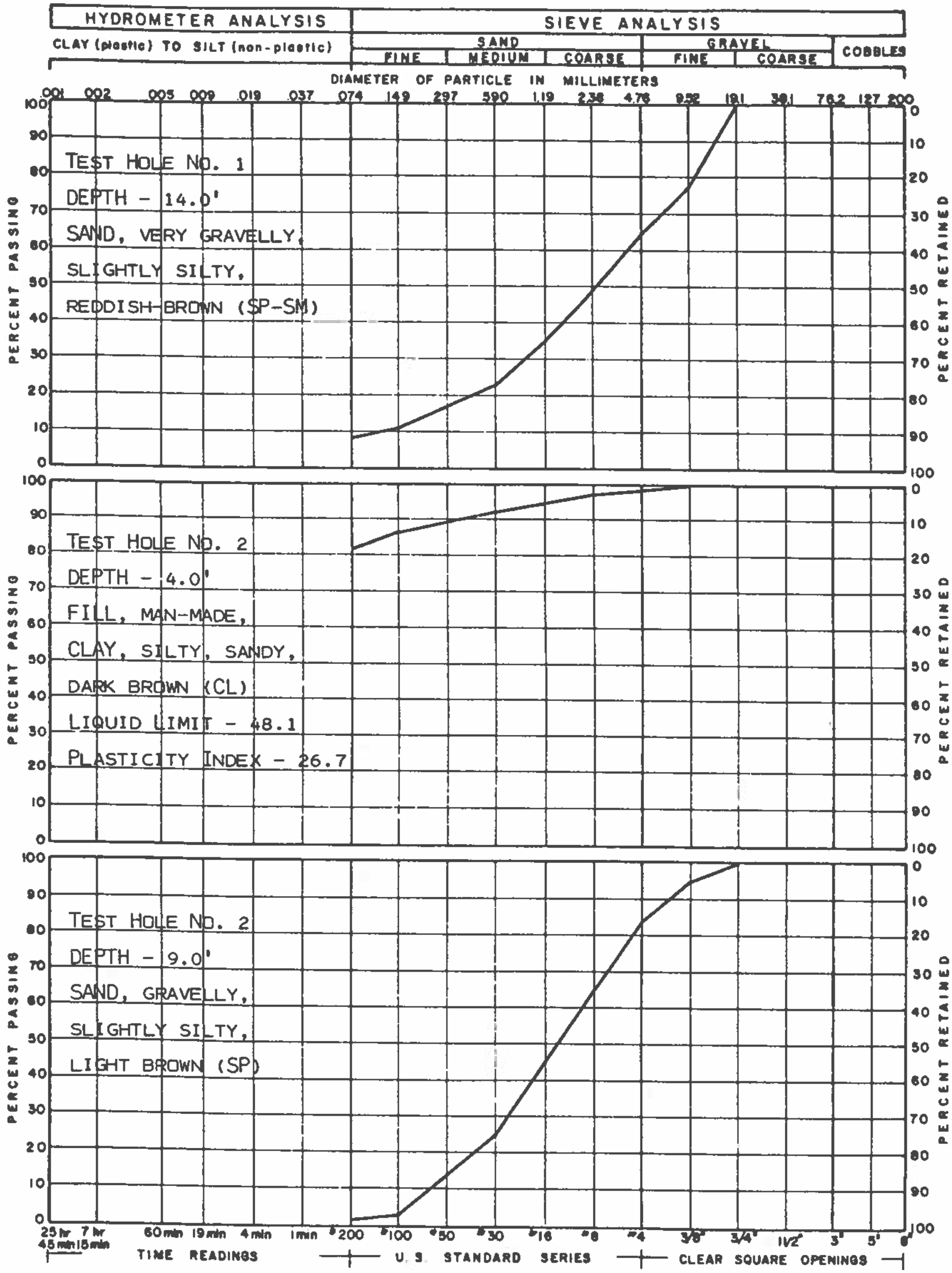

FIGURE 8

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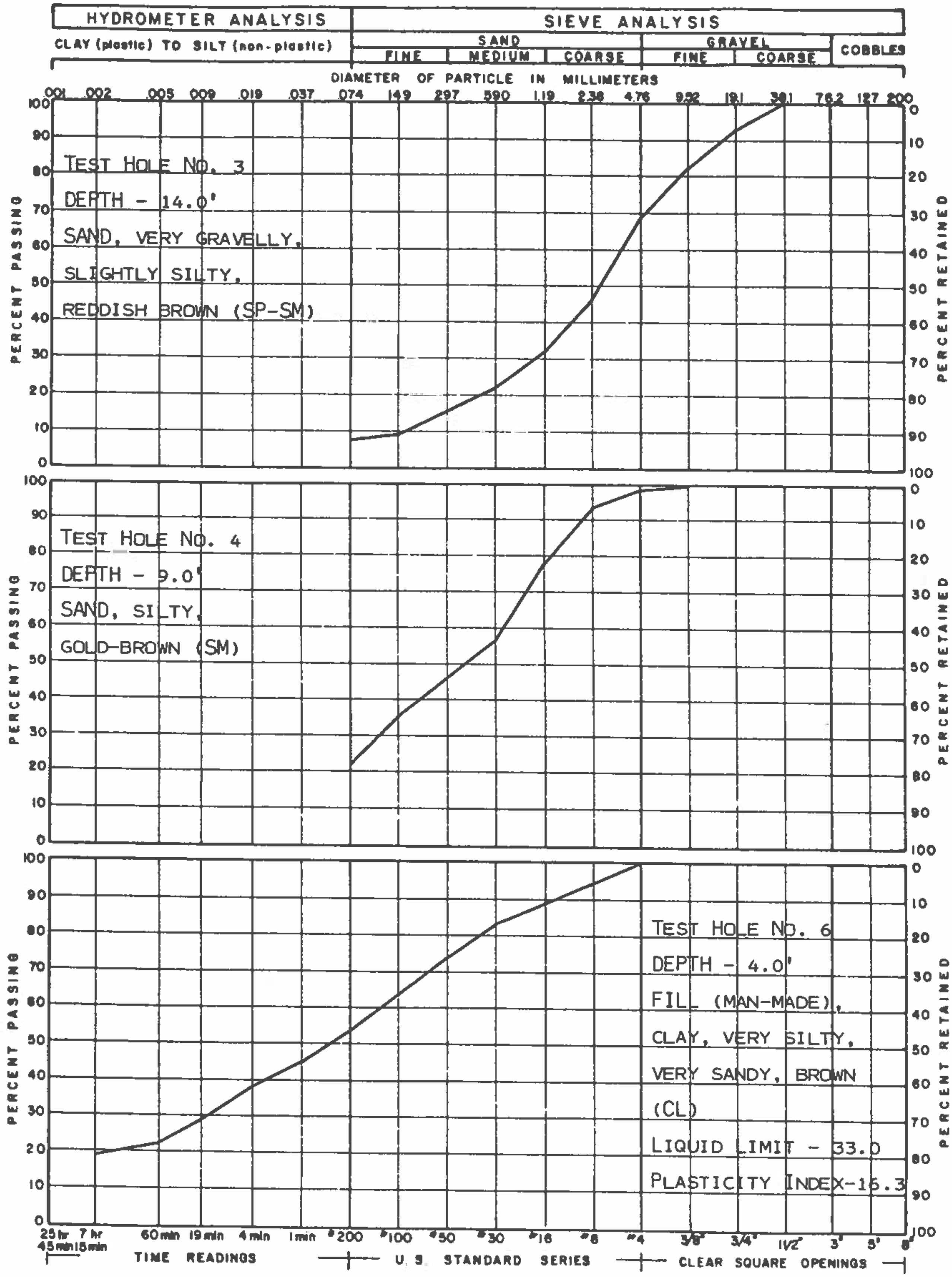
GRADATION TEST RESULTS



A. G. WASSENAAR, INC.
GEOTECHNICAL CONSULTANTS

GRADATION TEST RESULTS

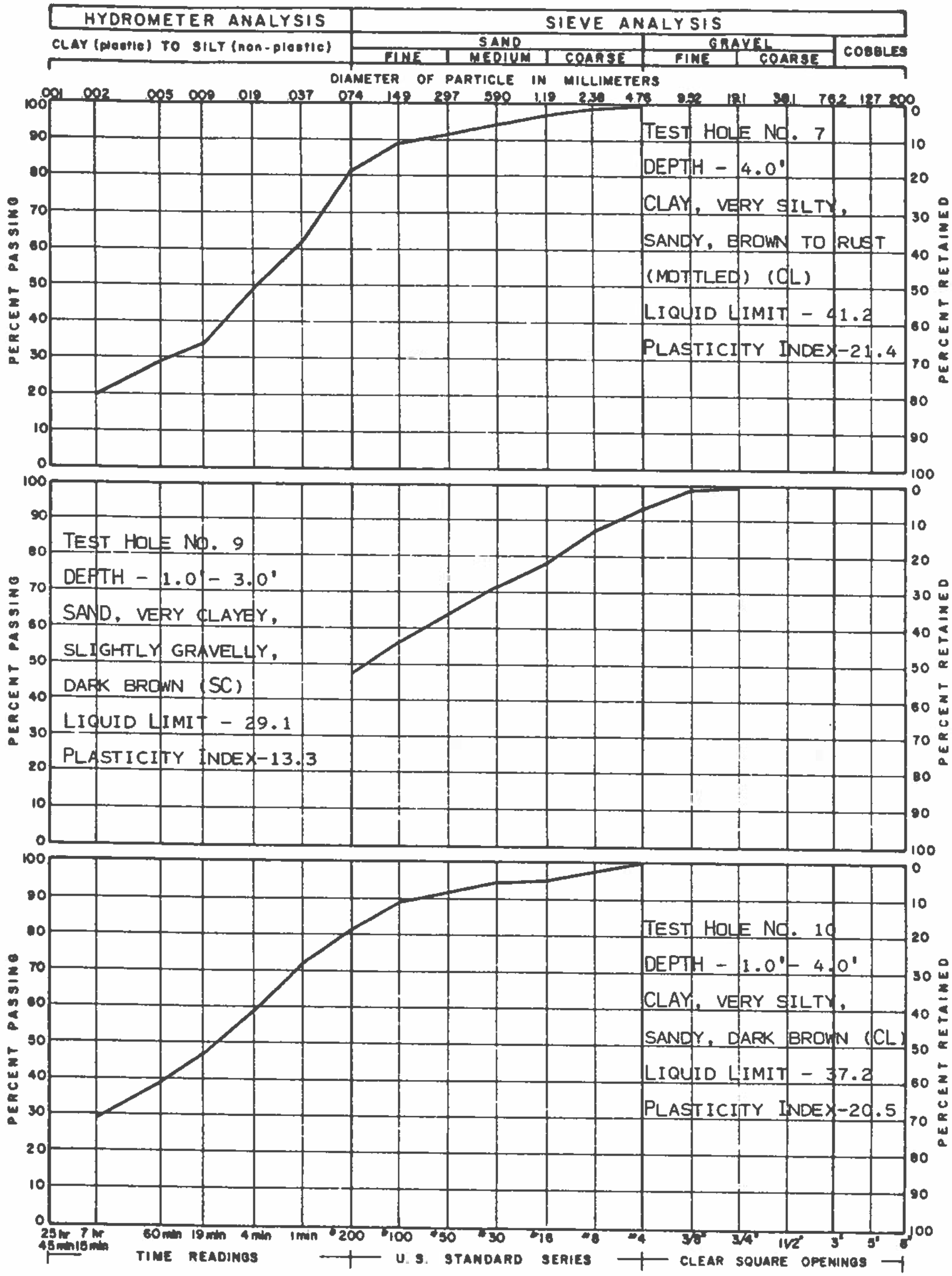


FIGURE 10

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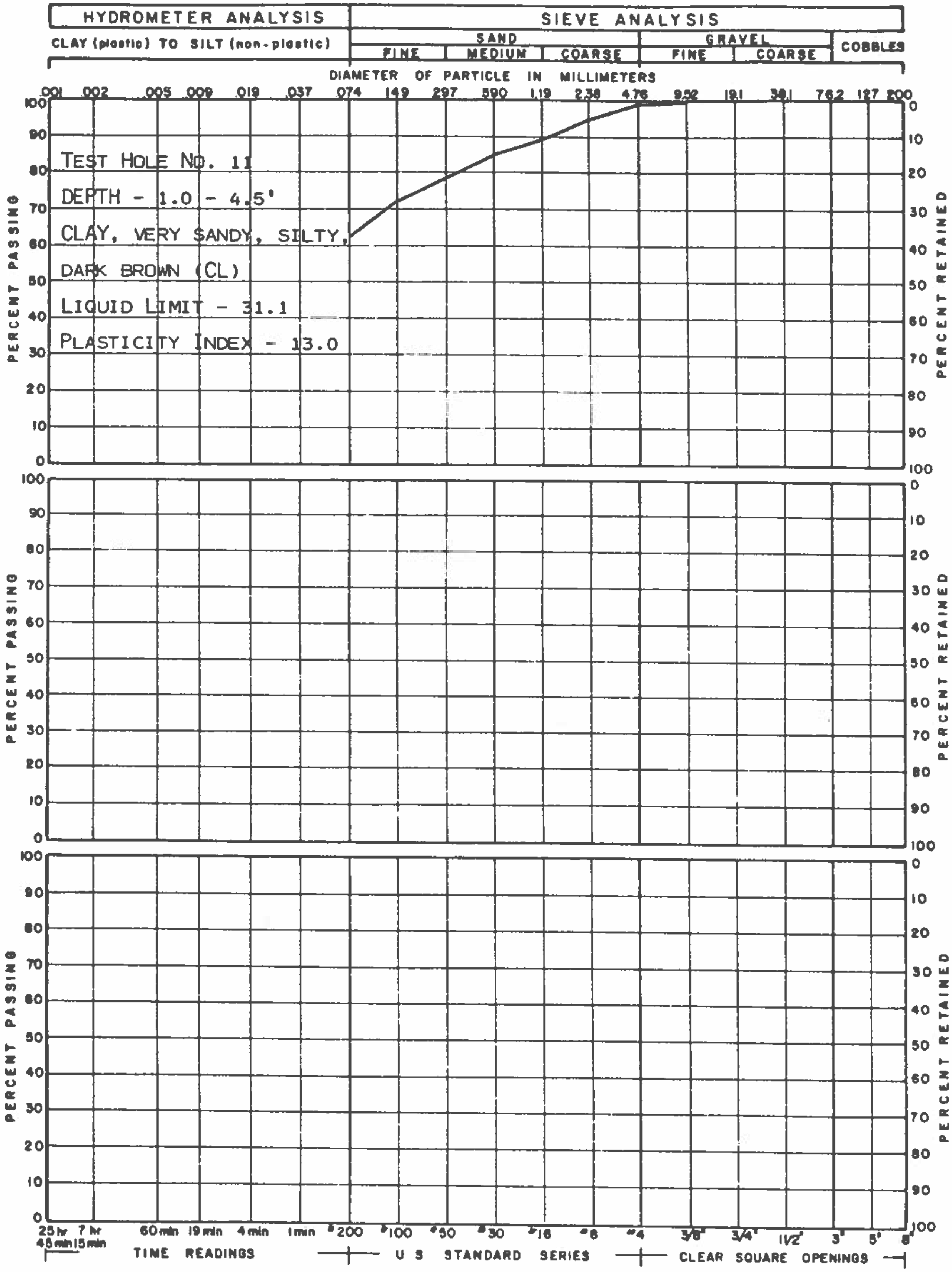
GRADATION TEST RESULTS



SPECIFICATIONS FOR PLACEMENT OF STRUCTURAL FILL

GENERAL

The Soil Engineer, as the Owner's representative, shall conduct tests to determine if the material, method of placement, and compaction are in reasonable compliance with the specifications.

MATERIALS

Granular material, well graded, having 100% finer than 3 inches in diameter and not more than 10% passing a No. 200 sieve, will be satisfactory for fill beneath footings provided the plastic index is zero. Soils not meeting the above specifications but proposed for fill should be tested and approved by a Soil Engineer. On-site clay and clayey sand soils will be suitable for fill beneath paving and as foundation backfill.

PREPARATION OF NATURAL GROUND

Vegetation, organic topsoil, and existing man-made fill shall be removed from the fill area. The area to be filled shall then be scarified, moistened if necessary, and compacted in the manner specified below for the subsequent layers of fill.

PLACEMENT OF FILL MATERIAL

No brush, sod, frozen, perishable, or other unsuitable material shall be placed in the fill. The materials shall be delivered to the fill in a manner which will permit a well and uniformly compacted fill. Before compacting, the fill material shall be spread in approximately horizontal layers not greater than 6 inches thick.

MOISTURE CONTROL

While being compacted, the material shall contain uniformly distributed optimum moisture for compaction. The Contractor shall be required to add moisture to the materials in the excavation if, in the opinion of the Soil Engineer, it is not possible to obtain proper and uniform moisture by adding water to the fill surface.

COMPACTION

When the moisture content and conditions of each layer spread are satisfactory, it shall then be compacted by an approved method. Compaction shall be at least 95% of maximum density for fill around the structures and beneath pavement and 100% for fill beneath bridge footings or box culverts. Moisture-density tests should be performed on typical fill materials to determine the maximum density. Field density tests must then be made to determine the adequacy of the fill compaction. The compaction standard to be utilized in determining the maximum density is ASTM D698. If the structural fill contains less than 10 percent passing the No. 200 sieve, it may be necessary to control compaction based on relative density (ASTM D2049). If this is the case, then compaction around the structures and beneath slabs and pavements shall be to at least 60% relative density, and compaction beneath foundations shall be to at least 70% relative density.

A. G. WASSENAAR, INC.
GEOTECHNICAL CONSULTANTS
COLORADO SPRINGS, COLORADO 80918 6755 EARL DRIVE, SUITE 105 303/590-9500

August 29, 1985

SELLARDS AND GRIGG, INC.
143 Union Boulevard
Suite 280
Lakewood, CO 80228

ATTENTION: MR. CHUCK McKNIGHT

SUBJECT: WATER SOLUBLE SULFATE TEST RESULTS
Channel Improvements
Cheyenne Creek/Meadows Park
Colorado Springs, Colorado
Project No. 85059

Gentlemen:

We have completed the water soluble sulfate tests for the subject project. These tests are conducted to determine the detrimental effect of the subsoils on concrete. The results are tabulated below:

Test Hole	Depth	% Water Soluble Sulfates
1	4.0'	0.001
4	4.0'	0.013
5	4.0'	0.007
8	4.0'	0.001

According to published information, no special cement is necessary for concrete which will be in contact with the soils.

If you have any questions concerning these test results, please call.

Sincerely,

A. G. WASSENAAR, INC.

COLORADO REGISTERED
MARK V. HERBERT
17895
PROFESSIONAL ENGINEER

Mark V. Herbert

MARK V. HERBERT, P. E.

MVH/dmw

Enclosures (4)