Table of Contents

File 1982-0049 Project Name: 6 &50 West Subdivsion – Filing #3 – Final Plat
Date 7/15/02

Present	Scanned	A few items are denoted with an asterisk (*), which means they are to be scanned for permanent record on the in some instances, not all entries designated to be scanned by the department are present in the file. There are also documents specific to certain files, not found on the standard list. For this reason, a checklist has been provided. Remaining items, (not selected for scanning), will be marked present on the checklist. This index can serve as a quick guide for the contents of each file. Files denoted with (**) are to be located using the ISYS Query System. Planning Clearance will need to be typed in full, as well as other entries such as Ordinances, Resolutions, Board of Appeals, and etc.
X	X	***Summary Sheet – Table of Contents**
X	X	**Review Sheet Summary**
		Application form
		Review Sheets
		Receipts for fees paid for anything
		***Submittal checklist**
X	X	***General project report**
		Reduced copy of final plans or drawings
		Reduction of assessor's map
		Evidence of title, deeds
X	X	***Mailing list to adjacent property owners**
		Public notice cards
		Record of certified mail
X		Legal description
		Appraisal of raw land
		Reduction of any maps – final copy
		***Final reports for drainage and soils (geotechnical reports)**
		Other bound or nonbound reports
		Traffic studies
		Individual review comments from agencies
		***Consolidated review comments list**
		***Petitioner's response to comments**
		***Staff Reports**
		***Planning Commission staff report and exhibits**
		***City Council staff report and exhibits**
		***Summary sheet of final conditions**
		***Letters and correspondence dated after the date of final approval (pertaining to change in conditions or expiration date)**

DOCUMENTS SPECIFIC TO THIS DEVELOPMENT FILE:

X	X	Action Sheet	X		Roadaway Profiles
X	X	Review Sheet Summary	X		Sanitary Sewer & Domestic Water Plans
X		Review Sheets	X		Domestic Water Plans
X		Subdivision Summary Form	X		Sanitary Sewer Profile
X	X	Planning Commission Minutes - ** - 7/27/82	X		Sanitary Sewer Details
X	X	Drainage Report	X		Location Map
X		Public Notice Posting – 7/13/82	X		Frontage Road Plans
X		Request for Treasurer's Certificate of Taxes Due – 6/22/82	X		Storm Drainage Plan & Profile
X		Development Application – 6/14/82	X		Drainage Ditch Plan & Profile
X		Subsurface Soil Investigation – 4/24/79	X		Grading & Drainage Plan
X		Deed			
X	X	Utilities Composite			
X	X	Letter from David Campbell, Dept. of Highways to James Preble, Paragon Eng., Inc. re: position on access for the Sub-7/26/82			

2945-151-00-016
Taylor, Robert Newton
2310 E. 1/2 Rd.
Grand Junction, CO. 81503

#49-82

2945-151-06-001
Derby, Jerry R.
360 W. Gunnison
Grand Junction, CO. 81501

#49-82

2945-151-10-011
J & J Enterprises
P. O. Box 2966
Grand Junction, CO. 81502

#49-82

Paragon Engineering
2284 Crossroads Blvd.
Grand Jct CO 81501

#49-82

2945-151-10-010
Hoaglund, Carl A. Etal
2654 Sacoma Ct.
Grand Junction, CO. 81501

#49-82

~~2945-151-11-001~~
~~002~~
~~003~~
~~004~~
~~005~~
~~006~~
~~007~~
~~008~~
~~018~~
Excalibur Enterprises
3033 S. Parker Rd. #602
Aurora, CO. 80014

#49-82

2945-152-00-007
Venegas Albino
P. O. Box 1883
Grand Junction, CO. 81502

#49-82

2945-152-00-006
Harbert Investments Co.
2245 Knollwood Ln.
Grand Junction, CO. 81501

#49-82

2945-152-00-002
Anna Company
P. O. Box 489
Grand Junction, CO. 81502

#49-82

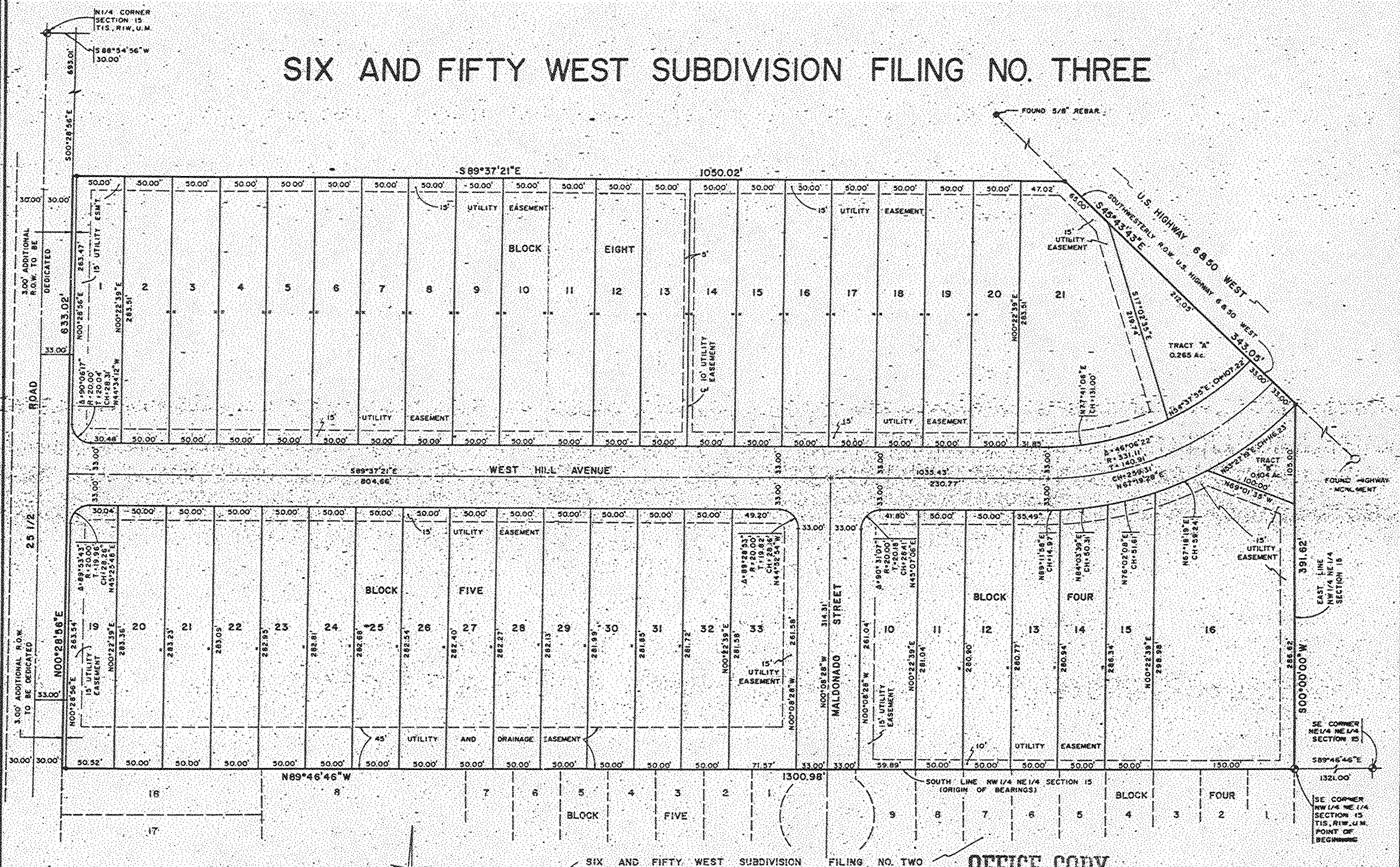
2945-151-00-096
Turtle Enterprises
P. O. Box 2477
Grand Junction, CO. 81502

#49-82

Six and Fifty West
Filing 3

Y

SIX AND FIFTY WEST SUBDIVISION FILING NO. THREE



OFFICE COPY

#4982

LEGEND

- INDICATES FOUND MESA COUNTY BRASS CAP
- INDICATES 5/8" REBAR AND MONUMENT CAP (LS# 9960) SET IN CONCRETE
- INDICATES 5/8" REBAR AND MONUMENT CAP (LS# 9960) SET AT ALL LOT CORNERS

0 50 100 150
SCALE 1"=50'

AREA QUANTITIES

TOTAL AREA IN LOTS	15.373 Acres	or 84.73%
TOTAL AREA IN TRACTS	0.369 Acres	or 2.03%
TOTAL AREA IN STREETS	2.402 Acres	or 13.24%
TOTAL AREA	18.144 Acres	or 100.00%

TOTAL NUMBER OF LOTS = 43

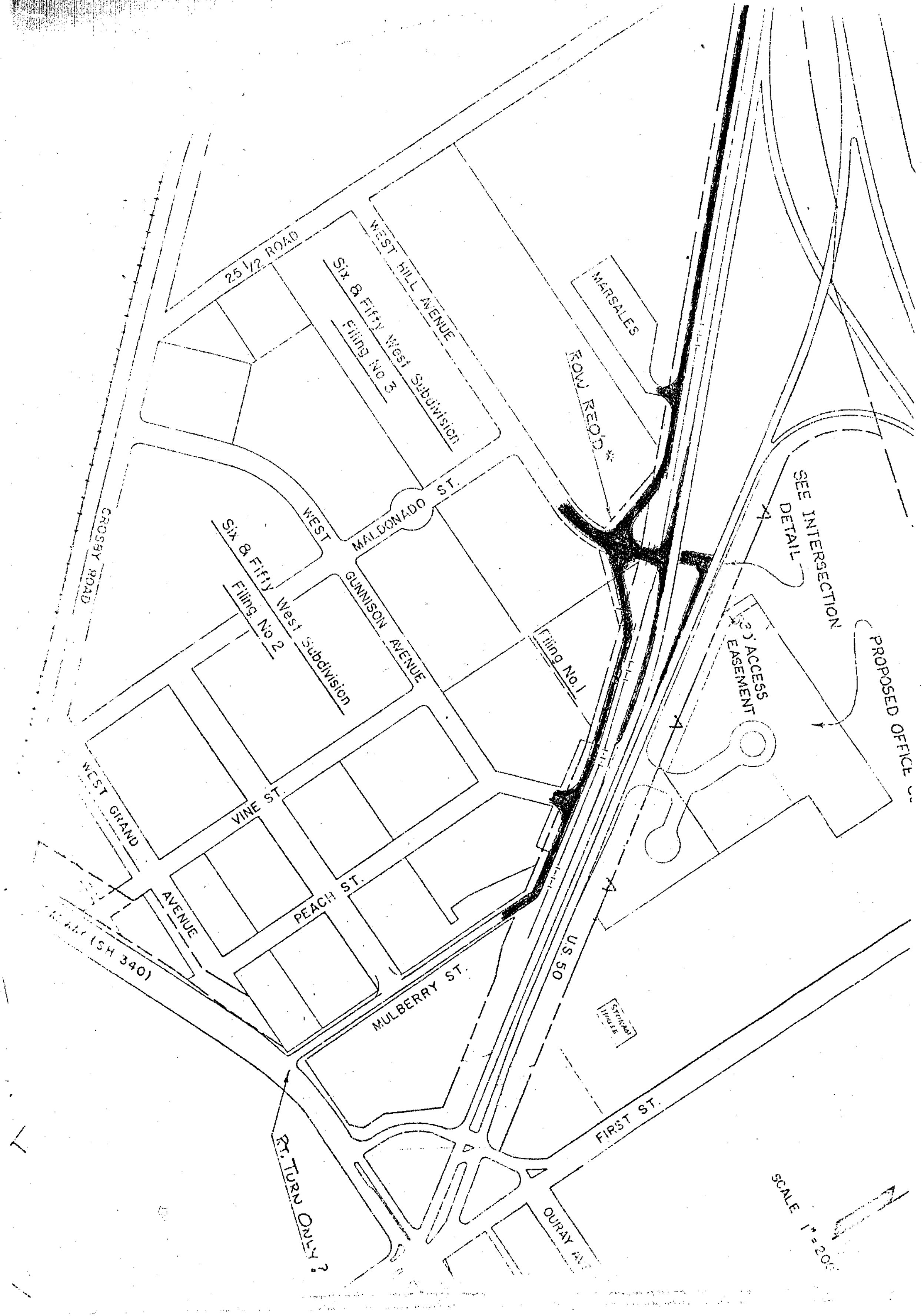
SHEET 2 OF 2

SIX AND FIFTY WEST SUBDIVISION FILING NO. THREE

PARAGON ENGINEERING, INC.

0535-010

EE



25 1/2 ROAD
WEST HILL AVENUE
MARSALES
Six & Fifty West Subdivision
Filing No 3
ROW REQ'D *
WEST MALDONADO ST.
SEE INTERSECTION DETAIL
PROPOSED OFFICE
CROSBY ROAD
Six & Fifty West Subdivision
Filing No 2
GUNNISON AVENUE
Filing No. 1
ACCESS EASEMENT
WEST GRAND AVENUE
VINE ST.
PEACH ST.
(SH 340)
MULBERRY ST.
US 50
FIRST ST.
RT. TURN ONLY?
OURAY AV
SCALE 1"=200

GREEN RIVER VERY FINE SANDY LOAM, DEEP OVER GRAVEL, 0 to 2 percent slopes, Class IIs Land (Gm)

This soil occurs along the Gunnison and Colorado Rivers, but for the most part at higher levels than the other Green River soils. Its better position makes it less susceptible to flooding or occasional high water tables. It can be cropped successfully, especially after it has been ditched to provide adequate underdrainage.

The surface soil, a pale-brown or light brownish-gray very fine sandy loam, contains numerous small fragments of mica. Below depths of 10 to 12 inches, the very fine sandy loam has a brighter pale-brown or very pale-brown color, and at depths of 24 to 30 inches it grades into similarly textured soil material that shows light-gray and reddish-brown specks or very small spots. Below depths of 3 or 4 feet textural variations are common, but fine sandy loam is dominant.

When moist, this soil is friable. Well-disseminated lime is present from the surface downward, but the organic-matter content is low. Workability and tilth are exceptionally favorable for irrigation and cultivation, but some places need ditches that will lower the water table.

Soil limitations are classified as severe for local roads and streets (seasonal high water tables, poor traffic-supporting capacity, subject to frost heave), shallow excavations (seasonal high water table), dwellings without basements (seasonal high water table), sanitary land fill (seasonal high water table), septic tank absorption fields (seasonal high water table), and sewage lagoons (rapid permeability below about 1 foot, seasonal high water tables.)

GREEN RIVER SILTY CLAY LOAM, DEEP OVER GRAVEL, 0 to 2 percent slopes, Group 20 II S1 (G1)

Normally this soil occurs on slightly lower levels than Green River fine sandy loam, deep over gravel, 0 to 2 percent slopes.

The surface soil, a pale-brown to light brownish-gray silty clay loam, extends to a depth of about 10 or 12 inches and grades into a very pale-brown or light brownish-gray silty clay loam. At depths of 18 to 26 inches small gray specks or faint mottlings are noticeable. Below 24 inches the soil consists of successive alluvial layers that vary in texture, depth and thickness. The entire profile is friable when moist.

Surface runoff and internal drainage are not adequate. Some areas that are exceptionally low and close to the river are affected by a high water table and by overflow in some years. Seepy places are prevalent in some areas. Most of the soil needs ditching or tiling to provide underdrainage, but so far the expense of obtaining proper drainage has been prohibitive. The soil contains considerable quantities of salts. Uncultivated areas, which account for approximately 90 percent of the acreage, are either moderately or severely saline. Soil tests indicate that lime is present in the surface soil and the subsoil.

Soil limitations are classified as severe for local roads and streets (moderately high water tables, poor traffic-supporting capacity, subject to frost heave), shallow excavations, dwellings with basements (high water tables, periodic flooding), dwellings without basements (high water tables, periodic flooding), sanitary land fill (occasional flooding, poorly drained), septic tank absorption fields (seasonal high water table), and sewage lagoons (rapid permeability below about 1 foot, moderately high water tables).

ENGINEER'S DRAINAGE REPORT

FOR

SIX AND FIFTY WEST SUBDIVISION

FILING NO. THREE

Prepared For

Albino Venegas
2429 H Road
Grand Junction, CO 81501

ROBERT P. GERLOFS
REGISTERED
9405
PROFESSIONAL ENGINEER
STATE OF COLORADO

ENGINEER'S DRAINAGE REPORT

SIX AND FIFTY WEST SUBDIVISION

FILING NO. THREE

SITE DESCRIPTION

The project is located in the Northeast 1/4 of Section 15, Township 1 South, Range 1 West of the Ute Meridian, City of Grand Junction, County of Mesa, State of Colorado, containing 18.14 acres.

The site is currently 1/2 open range land with approximately the West 1/2 as irrigated farm land. The land historically drains to the Northwest at an approximate slope of 1% to an earthen drainage channel located approximately at the projected intersection of 25-1/2 Road and Highway 6 & 50.

The site is bordered on the South by "Six and Fifty Subdivision Filing No. Two", North and West by undeveloped land and the East by Highway 6 and 50.

HISTORICAL DRAINAGE

The historic drainage basin draining onto the site contains 45.6 acres. The projected runoff from this basin, based on the factors from the drainage calculations from "Six and Fifty Subdivision Filing No. Two", is as follows:

A	=	45.6 Acres			
C	=	0.75			
L	=	2,600 Ft.	S	=	1.0%
Δh	=	26 Ft.			
t_c	=	32 Min.			
Q_{10}	=	47.9 cfs			
Q_{100}	=	94.1 cfs			

The 10 year runoff is intercepted by a 90 foot drainage swale along the Southerly border of the project with a capacity of 66.5 cfs and does not affect the site.

The historic drainage for the project by itself is as follows:

A	=	18.14 Acres			
C	=	0.35			
L	=	1,450 Ft.	S	=	0.4%
Δh	=	6 Ft.			
t_c	=	70 Min.			
Q_{10}	=	6.5 cfs			
Q_{100}	=	9.0 cfs			

The historic drainage, as previously stated, flows Northwest into an existing earthen drainage swale.

The site is not affected by the 100 Year Flood Plain as shown on Plate 22 of the "Flood Hazard Information" Grand Junction, Colorado, as prepared by the Corps of Engineers, November, 1976.

The subsurface soil investigation was prepared by Lincoln-DeVore, Job No. J-228.

DRAINAGE CRITERIA

The criteria used to evaluate this development is that outlined in "Design Guidelines for Storm Water Management" in Mesa County, Colorado and "Urban Storm Drainage Criteria Manual" published by Denver Regional Council of Government (D.R.C.O.G.).

The rational method was used to calculate the peak developed flows for the design storm (10 year) and the major flood storm (100 year). The rational formula $Q = (C\ C_c)\ I\ A$ was used where:

Q	=	Storm Flow (cfs)
I	=	Rainfall Intensity (inches per hour)
A	=	Drainage Basin (acres)
C	=	Runoff Coefficient
C_f	=	Storm Frequency Factor

The following runoff coefficients "C" were used to calculate the runoff:

Historic	Unimproved	0.30
	Irrigated	0.40
Developed	Industrial	0.75

The time of concentration was developed using Overland Flow Charts and the formula $t_c = \frac{1.8\ (1.1 - C)\ \sqrt{D}}{\sqrt[3]{S}}$

The intensity is taken from the Intensity Duration Curves of Mesa County.

PROPOSED DEVELOPMENT AND DRAINAGE

The project is a proposed Industrial Park to be laid out as shown on the attached Grading and Drainage Plan. The grading will be as shown on the aforementioned plan creating 11 sub-basins as shown. The runoff calculations and discharge points for each basin are stated in the Appendix of this report.

A drainage easement containing a concrete swale with a carrying capacity of 18.2 cfs will intercept and convey the runoff through the project.

The total offsite and generated runoff from the 10 year storm will be carried via a concrete swale in the Easterly Right-of-Way of 25-1/2 Road and dedicated easements to the historical discharge location.

All finished floors of proposed structures should have a minimum of 1.5 foot free-board above the closest drainage structure.

The 100 year runoff will be transferred via the proposed Right-of-Ways and drainage structures and will not create any projected adverse conditions downstream.

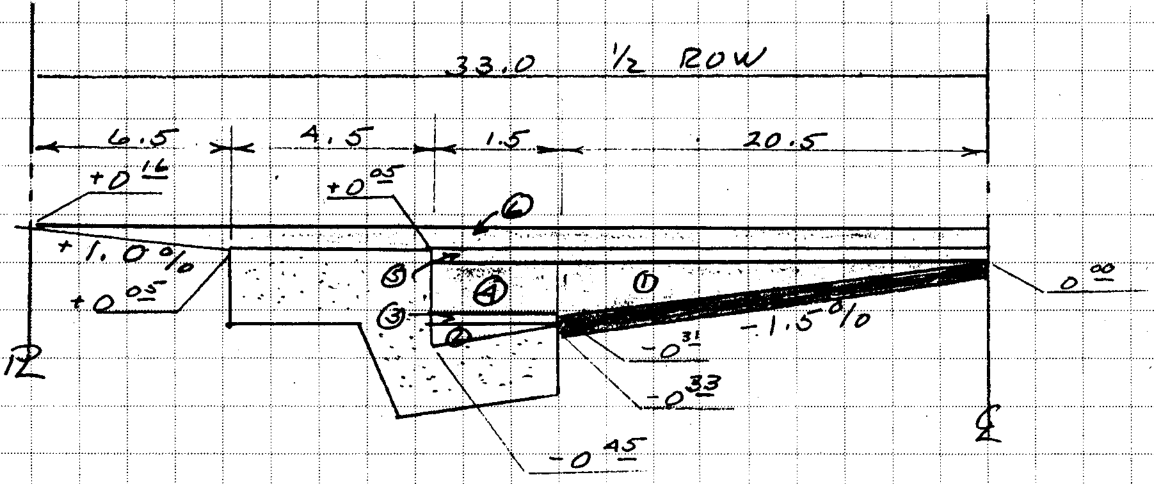
I hereby certify that this Report was prepared under my direct supervision for the Owner thereof.

Robert P. Gerlofs
Registered Professional Engineer
Colorado Registeration No. 9402

PARAGON ENGINEERING, INC.
2784 Crossroads Blvd. Suite 104
GRAND JUNCTION, COLORADO 81501
(303) 243-8966

JOB 6 & 50 West #3
SHEET NO. ______ OF ______
CALCULATED BY J.W.P. DATE 6/82
CHECKED BY ______ DATE ______
SCALE ______

STREET CAPACITY



AREA

① $A = \frac{1}{2}(20.5)(0.31) = 3.18$

② $A = \frac{1}{2}(1.5)(0.12) = 0.09$

③ $A = (1.5)(0.02) = 0.03$

④ $A = (1.5)(0.31) = 0.47$

⑤ $A = (0.05)(22.0) = 1.10$

⑥ $A = \frac{1}{2}(33.0 + 26.5)(0.11) = 3.27$

$\Sigma_T \tfrac{1}{2} A = 7.04 \qquad \Sigma_T = 14.08$ S.F.

$A = 14.08$ SF $\qquad R = 0.213$ FT $\qquad S = 0.3\%$

$WP \approx 66.2$ FT

$n = 0.016$

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

$$Q = \frac{1.486}{0.016}(14.08)(0.213)^{2/3}(0.003)^{1/2} = 25.5 \text{ cfs}$$

MAX CAPACITY $Q = 25.5$ cfs

(WITH IN R.O.W.)

MAX VELOCITY $V = 1.81$ fps

PARAGON ENGINEERING INC.
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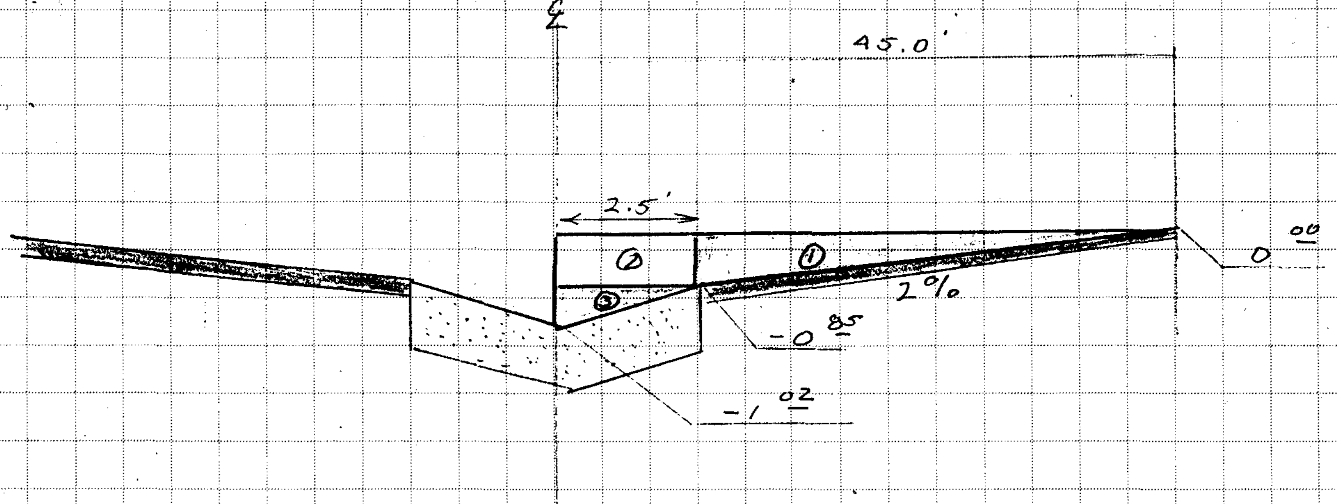
JOB 6 & 50 West 3
SHEET NO. ____ OF ____
CALCULATED BY J. W. P. DATE 6/82
CHECKED BY ____ DATE ____
SCALE ____

90 FT SWALE CAPACITY



$A_1 = \frac{1}{2}(42.5)(0.85) = 18.06$

$A_2 = (2.5)(0.85) = 2.13$

$A_3 = \frac{1}{2}(2.5)(0.17) = 0.21$

$\Sigma_T \frac{1}{2} A = 20.40$ $\Sigma_T A = 40.80$

$A = 40.80$ $R = 0.453$ $S = 0.1\%$

$WP = 90.03$

$n = 0.017$

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

$$Q = \frac{1.486}{0.017}(40.8)(0.453)^{2/3}(0.001)^{1/2}$$

$$Q = 66.54 \text{ cfs}$$

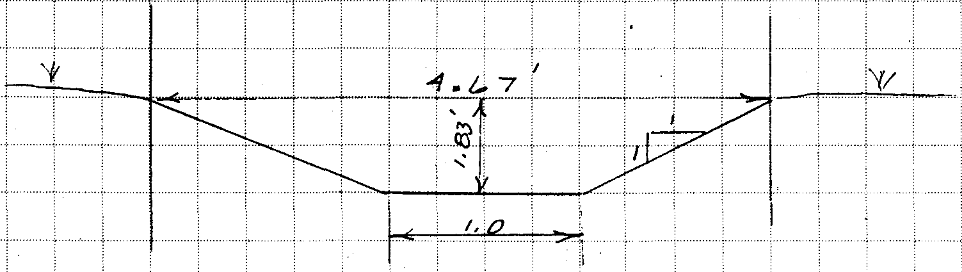
MAX CAPACITY 66.5 cfs
MAX VELOCITY 1.63 fps

PRODUCT 204-1 NEBS Inc., Groton, Mass. 01471.

PARAGON ENGINEERING INC.
2784 Crossroads Blvd. Suite 104
GRAND JUNCTION, COLORADO 81501
(303) 243-8966

JOB 6 & 50 West ~3
SHEET NO. ______ OF ______
CALCULATED BY J. W. P. DATE 6/82
CHECKED BY ______ DATE ______
SCALE ______

CHANNEL CAPACITY
INTERIOR



$$A = \left(\frac{4.67+1.0}{2}\right)(1.83) = 5.19 \text{ SF}$$

WP = 6.17 FT

R = 0.84 FT

S = 0.17 %

USE n = 0.013 (concrete)

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

$$Q = \frac{1.486}{0.013} (5.19)(0.84)^{2/3} (0.00168)^{1/2} = 21.65$$

MAX CAPACITY 21.65 cfs
MAX VELOCITY 4.17 fps

PARAGON ENGINEERING, INC.
2784 Crossroads Blvd. Suite 104
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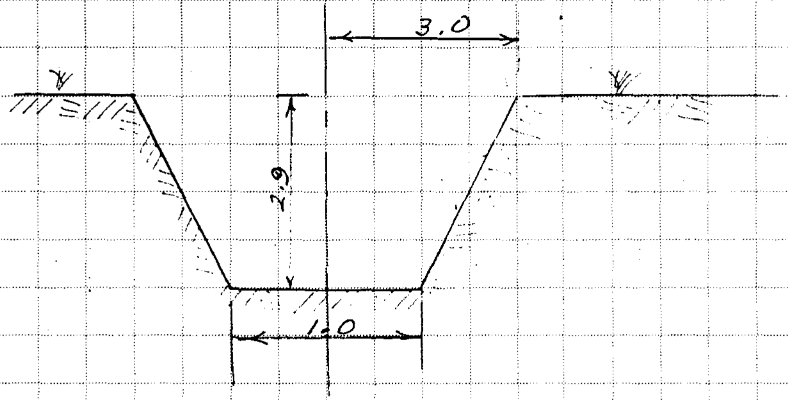
(303) 243-8966

JOB 6 & 5[illegible] West #3
SHEET NO. ____ OF ____
CALCULATED BY J. W. P. DATE 6/82
CHECKED BY ____ DATE ____
SCALE ____

CHANNEL CAPACITY
25 1/2 ROAD



$A = \left(\frac{6+1}{2}\right)(2.9) = 10.15$ SF

WP = 8.16 SF

R = 1.17 Ft

S = 0.36 %

use $n = 0.013$

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

$$Q = \frac{1.486}{0.013} (10.15)(1.17)^{2/3}(0.0036)^{1/2} = 77.4$$

MAX CAPACITY = 77.4 cfs
MAX VELOCITY = 7.62 fps

SUBDIVISION - SIX & FIFTY #3
LOCATION
DESIGN STORM 10 YR RECURRENCE INTERVAL
COMPUTATIONS BY JWP DATE 6-10-82
SUBMITTED BY PARAGON DATE 6-10-82
(Engineering Firm)

STORM DRAINAGE SPECIFICATIONS

RUNOFF COMPUTATIONS
(Rational Method)

Curb Information

$Tc = \frac{1.8\,(1.1 - C)\,D^{1/2}}{S^{1/3}}$

Design Point	Area Designat.	A (Acres)	c	cf	$\bar{c}$= (c·cf)	A·$\bar{c}$	ΣA·$\bar{c}$	tc (min)	I (in/hr)	Q (ΣA·$\bar{c}$)·I cfs	Slope (S)	Length L (feet)	VEL V fps	Depth (Ft)	Time of Concentration
1	D	0.19	.75	1	0.75	0.14	0.14	10	2.55	0.36	0.94	125	0.21	0.10	10 min
Σ_T @ 1	+ OFFSITE				Q_{10} = 47.88					48.24					
2	A	3.72			0.75	2.79	2.79	20	1.8	5.02	0.5	660	.55	0.23	20 min
3	E	0.81			0.75	0.61	3.40	21	1.75	5.95	0.3	850	0.67	0.24	21 min
3	F	1.68			0.75	1.26	1.26	18	1.9	2.39	0.3	530	0.49	0.18	18 min
Σ_T @ 3										8.34					
4	B	2.89			0.75	2.17	2.17	20	1.8	3.91	0.3	600	0.50	0.21	20 min
4	K	1.54			0.75	1.16	1.16	16	2.1	2.44	0.3	400	0.42	0.18	16 min
Σ_T @ 4										14.69					
6	G	1.98			0.75	1.49	1.49	22	1.7	2.53	0.3	900	0.68	0.18	22 min
Σ_T @ 6				+ Σ_T @ 1		Q = 48.24				50.77					

SUBDIVISION - 5TH & FIFTY #3
LOCATION
DESIGN STORM 10 YR RECURRENCE INTERVAL
COMPUTATIONS BY JWP DATE 6-10-82
SUBMITTED BY PARAGON DATE 6-10-82
(Engineering Firm)

STORM DRAINAGE SPECIFICATIONS

RUNOFF COMPUTATIONS
(Rational Method)

Curb Information

$Tc = \frac{1.8\,(1.1 - C)\,D^{1/2}}{S^{1/3}}$

Design Point	Area Designat.	A (Acres)	c	cf	$\bar{c}$= (c·cf)	A·$\bar{c}$	ΣA·$\bar{c}$	tc (min)	I (in/hr)	Q (ΣA·$\bar{c}$)·I cfs	Slope (S)	Length L (feet)	VEL V fps	Depth (Ft)	Time of Concentration
Σ_T @ 6										50.77					
7	H	1.63	.75	1.0	0.75	1.22	1.22	22	1.7	2.07	0.3	480		0.17	22 min
Σ_T @ 7										52.84					
8	J	1.13			0.75	0.85	0.85	22	1.7	1.44	0.3	480		0.15	22 min
Σ_T @ 8										54.28					
5	C	0.82			0.75	0.61	0.61	16	2.1	1.28	0.3	250			16 min
Σ_T @ 5		+ Σ_T @ 4				Q =	14.69			15.97					
9	L	1.75			0.75	1.31	1.31	22	1.7	2.23	0.3	900			22 min
		+ Σ_T @ 8				Q =	54.28			54.28					
Σ_T @ 9										72.48					

SUBDIVISION ____________________
LOCATION ____________________
DESIGN STORM 100 YR RECURRENCE INTERVAL
COMPUTATIONS BY JWP DATE ______
SUBMITTED BY PARAGON DATE ______
(Engineering Firm)

STORM DRAINAGE SPECIFICATIONS

RUNOFF COMPUTATIONS
(Rational Method)

Curb Information

$T_c = \frac{1.8\,(1.1 - C)\,D^{1/2}}{S^{1/3}}$

Design Point	Area Designat.	A (Acres)	c	cf	$\bar{c}$ = (c·cf)	A·$\bar{c}$	ΣA·$\bar{c}$	tc (min)	I (in/hr)	Q (ΣA·$\bar{c}$)·I cfs	Slope (S)	Length L (feet)	VEL V fps	Depth (Ft)	Time of Concentration
1	D	0.19	.75	1.25	0.94	0.18	0.18	10	3.9	0.70					
Σ_T @ 1		+	OFFSITE			Q =	94.05			94.75					
2	A	3.72			0.94	3.49	3.49	20	2.8	9.77					
3	E	0.81			0.94	0.76	4.25	21	2.75	11.69					
3	F	1.68			0.94	1.58	1.58	18	3.0	4.74					
Σ_T @ 3										16.43					
4	B	2.89			0.94	2.71	2.71	20	2.8	7.59					
4	K	1.54			0.94	1.44	1.44	16	3.15	4.55					
Σ_T @ 4										28.57					
6	G	1.98			0.94	1.86	1.86	22	2.7	5.02					
Σ_T @ 6		+ Σ_T @ 1				Q = 94.75				99.77					

SEE 10 YEAR STORM

UBDIVISION ____
OCATION ____
ESIGN STORM 100 YR RECURRENCE INTERVAL
OMPUTATIONS BY JWP DATE ____
UBMITTED BY PARAGON DATE ____
(Engineering Firm)

STORM DRAINAGE SPECIFICATIONS

RUNOFF COMPUTATIONS
(Rational Method)

											Curb Information				$T_c = \frac{1.8(1.1-C)D^{1/2}}{S^{1/3}}$
Design Point	Area Designat.	A (Acres)	c	cf	$\bar{c}$= (c·cf)	A·$\bar{c}$	Σ A·$\bar{c}$	tc (min)	I (in/hr)	Q (ΣA·$\bar{c}$)· I cfs	Slope (S)	Length L (feet)	VEL V fps	Depth (Ft)	Time of Concentration
Σ_T @ 6										99.77					
7	H	1.63	.75	1.25	0.94	1.53	1.53	22	2.7	4.13					
Σ_T @ 7										103.90					
8	J	1.13			0.94	1.06	1.06	22	2.7	2.86					
Σ_T @ 8										106.76					
5	C	0.82			0.94	0.77	0.77	16	3.15	2.42					
Σ_T @ 5				+ Σ_T	@	4	Q= 28.57			30.99					
9	L	1.75			0.94	1.64	1.64	22	2.7	4.43					
			+	Σ_T	@	8	Q: 106.76			106.76					
Σ_T @ 9										111.19					

SEE 10 YEAR STORM

$$\frac{Z}{n} = \frac{\frac{1}{0.015}}{0.02} = 3333$$

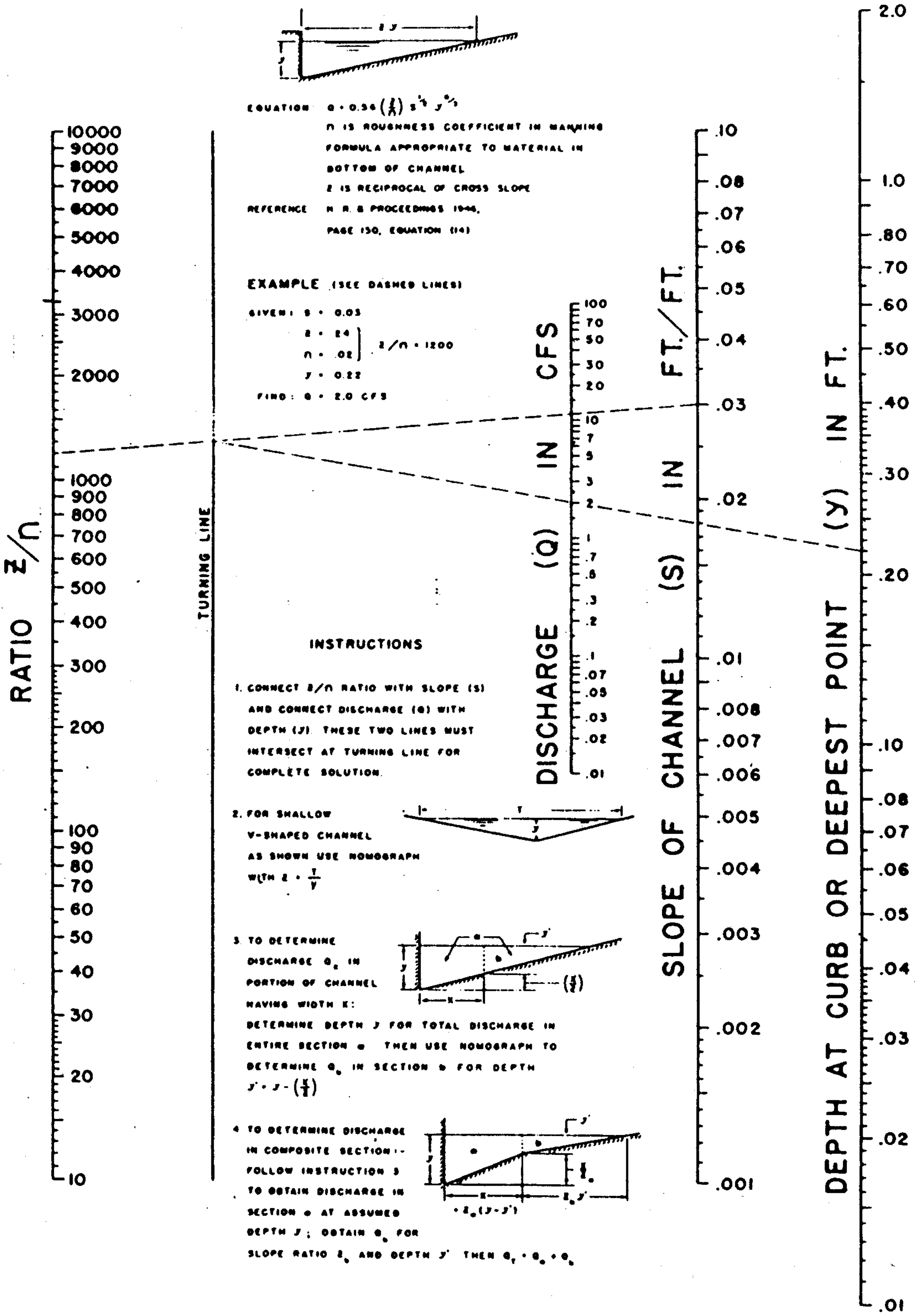
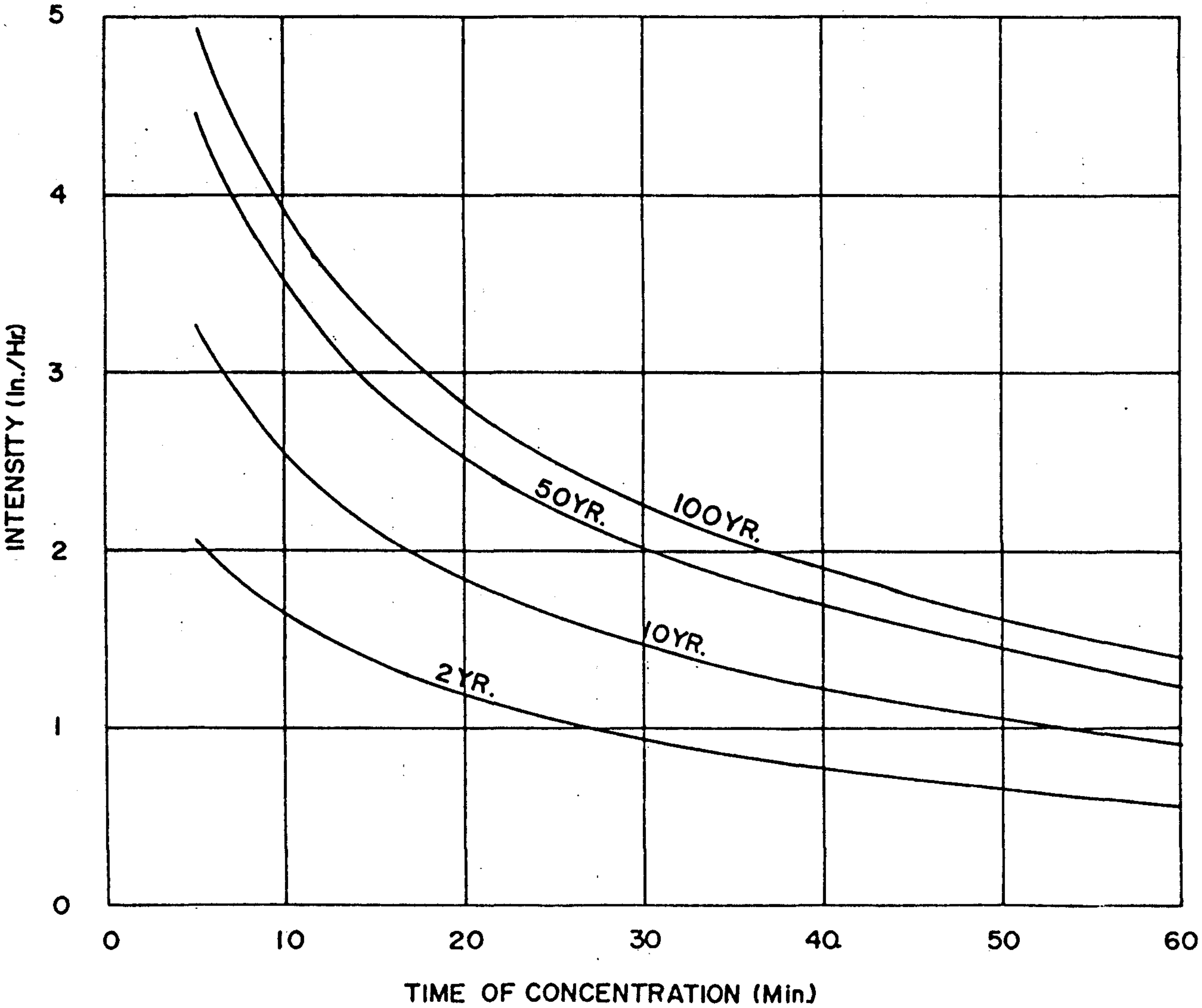
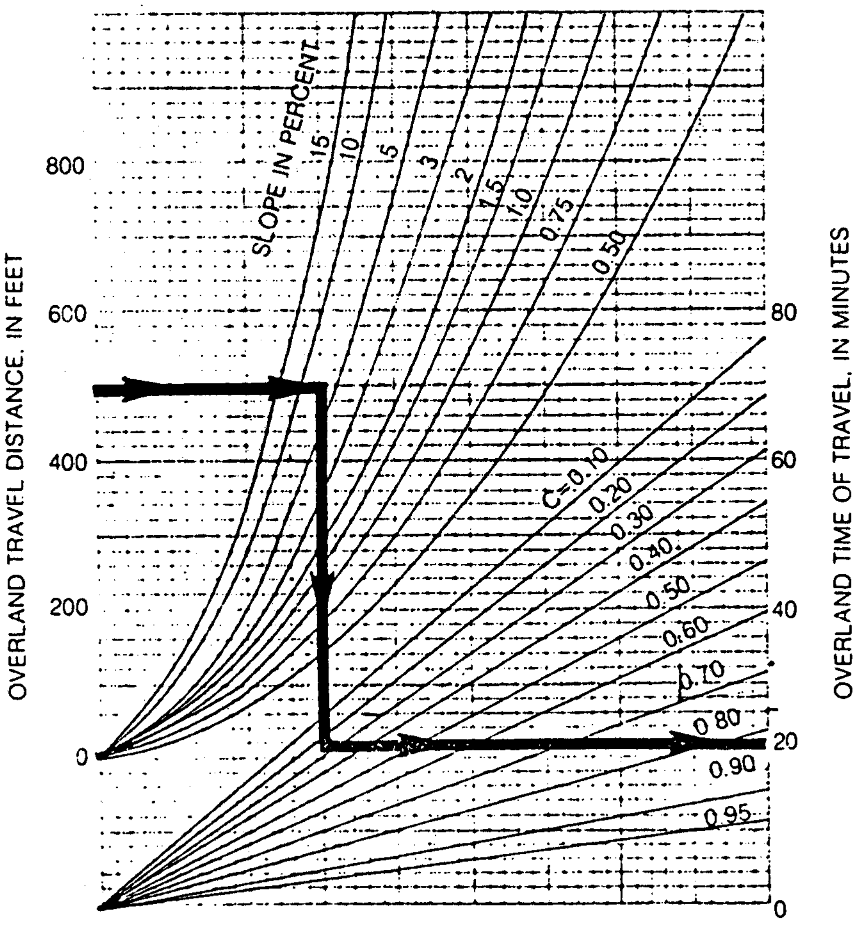


FIGURE 6-1. NOMOGRAPH FOR FLOW IN TRIANGULAR GUTTERS.



INTENSITY DURATION CURVES
GRAND JUNCTION, COLORADO



Relation of overland time of travel to overland travel distance, average overland slope, and coefficient C —for use in Rational Method.

Lincoln DeVore

1000 West Fillmore St.
Colorado Springs, Colorado 80907
(303) 632-3593
Home Office

May 24, 1979

Excalibur Enterprises, Inc.
P.O. Box 2266
Grand Junction, Colorado 81501

Re: SUBSURFACE SOILS INVESTIGATION

SIX AND FIFTY WEST
INDUSTRIAL SUBDIVISION

GRAND JUNCTION, COLORADO

Gentlemen:

Transmitted herewith is the report giving the results of a subsurface soils investigation for the proposed Six and Fifty West Industrial Subdivision in Grand Junction, Colorado.

Respectfully submitted,

LINCOLN-DeVORE TESTING LABORATORY, INC.

By: Robert L. Bass
Robert L. Bass
Civil Engineer

George D. Morris
Reviewed By George D. Morris, P. E.

RLB/vfb
LDTL Job No. J-381
cc: Paragon Engineers
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Grand Junction, CO 81501

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ABSTRACT

The contents of this report are a subsurface soils investigation and foundation recommendations for the proposed Six and Fifty West Industrial Subdivision Filing 3, to be located in the western portion of Grand Junction, Colorado. The Laboratory has not at this time seen a set of construction drawings for any of the structures to be constructed in this subdivision.

In instances where foundation loads are relatively light and material of suitable bearing capacity is available at relatively shallow depth, shallow foundation systems will probably be most suitable. Shallow foundations typically would consist of continuous foundations beneath bearing walls and isolated spread footings beneath columns and other points of concentrated load. The shallow foundation bearing capacity will be dependent upon the nature of the materials and will be variable from point to point. As an example, the denser materials located below the upper fine grained silt in Test Borings 1, 2, and 4 should be capable of developing bearing capacities of at least 3000 psf with no minimum pressure required. The bottoms of shallow foundations should be located at a minimum of 2 feet below finished grade or greater if dictated by local building codes, for frost protection.

In situations where the foundation loads are relatively heavy and where low density materials exist in the upper portion of the soil profile, a deep foundation system consisting of either driven piles or drilled piers will probably be most suitable. If either drilled piers or driven piles are used, they should be made to penetrate any overlying low density materials and rest in the underlying dense gravel and cobble deposits.

Some difficulty may be encountered in the installation of drilled pier deep foundation systems, due to soft caving soils and ground water problems, and therefore, casing and dewatering techniques may be necessary.

More complete recommendations can be found within the body of this report. All recommendations are subject to the limitations set forth herein.

GENERAL

The purpose of this investigation was to determine the general suitability of this site as an industrial subdivision. The Laboratory has not at the present time seen a set of construction drawings for any of the structures to constructed in this development. It is assumed that there is to be a variety of industrial type structures, using various construction techniques; and foundation loads, generally, will be moderate to high in magnitude. Characteristics of the individual soils encountered in the test borings were examined for use in designing foundations for these structures.

The construction site is located in the western portion of the city of Grand Junction, Colorado, between Highway 6 and 50, and the Colorado River. The Colorado River is located less than 1/2 mile to the southwest of the site. The site is in the northeast quarter of Section 15, Township 1 South, Range 1 West of the Ute Principal Meridian. This location is shown on the enclosed site location map.

The topography of this site is relatively flat, being located on an alluvial plain of the Colorado River. The area in the vicinity of the site has a slight gradient to the southwest towards the river. The exact direction of surface runoff will be controlled to an extent by streets and buildings to be constructed in this development and therefore, will be variable. In general, however, surface runoff will travel to the southwest quickly entering the Colorado River. Surface and subsurface drainage are poor.

The soils on this site are alluvial in nature, having been deposited on this site by the action of the Colorado River in the past. The soil profile can broadly be described as a layer of fine grain, silt material overlying an alluvial gravel and cobble deposit. The upper silt materials were generally in a low density, moist condition. The underlying gravelly materials were in a high moisture condition but were of variable density. In certain instances the upper portion of these gravelly materials was noted to be very loose. These materials increased in density, however, with depth and relatively dense material was encountered in all test borings within 15 feet of the ground surface.

These upper alluvial materials are believed to have been deposited on dense formational shale of the Mancos Formation. The Mancos Formation will serve as bedrock beneath this site. The Mancos Formation can broadly be described as thin-bedded, drab, light to dark grey marine shale, with thinly-interbedded fine grain sandstone and limestone layers. Some portions of the Mancos Shale are bentonitic and therefore, are highly expansive. The majority of the shale, however, has only a moderate expansion potential. The Mancos Shale was not encountered in any of the test borings placed on this site. Shale is believed to exist at sufficient depth that it will not effect construction or performance of the proposed foundation systems.

BORINGS, LABORATORY TESTS AND RESULTS

Six test borings were placed on this site at locations indicated on the enclosed Test Boring Location Diagram. These test borings were located in such a manner as to obtain a reasonably good profile of the subsurface soils. While some variation was noted from point to point sufficient information was obtained that no further test borings were deemed necessary. All test borings were advanced with a power-driven continuous auger drill. Samples were taken with the standard split-spoon sampler with thin-walled Shelby tubes and by bulk methods.

The soil profile encountered on this site can broadly be described as a two layer system. The upper layer of this system consisted of a low density, moist, fine grain silt material. This silt material is alluvial in nature, having been deposited in the past by overbank wash of the Colorado River. It was encountered in all test borings placed on this site, and varied in thickness from 2 to 5 feet. The second layer of the soil profile, which was encountered immediately below the previously described fine silt material, was a deposit of gravel and cobbles which are alluvial in nature, having been deposited by the action of the Colorado River in the past. These materials ranged in density from low to very high. Generally, the density of these materials was noted to increase with depth.

The samples obtained during our field exploration program have been grouped into three soil types. Soil Type No. 1 is representative of the fine grain silts of the upper layer of the soil profile. Soil Type No. 2 is representative of the alluvial gravel materials of the second layer of the soil profile. Soil Type No. 3 was a coarse grained, sandy silt material which was encountered in a low density condition in the lower portion of Test Boring No. 5. More precise engineering characteristics of these three soil types are given on the enclosed Summary Sheets. The following discussion will be general in nature.

Soil Type No. 1 classified as silt (ML) of generally fine grain size. This material is non-plastic, of low permeability, and was encountered in a low density condition. It will have virtually no tendency to expand upon the addition of moisture. It will, however, have a distinct tendency to long-term consolidation under load. It will also have a very low bearing capacity and will exhibit significant amounts of settlement upon application of foundation stresses. Therefore, it is recommended that foundations not rest in this material, but rather penetrate to the underlying coarser grained alluvium. Soil Type No. 1 contains sulfates in detrimental quantities.

Soil Type No. 2 classified as poorly graded gravel (GP) of coarse grain size. This material contained numerous cobble-sized particles which obviously cannot be accurately represented on the enclosed grain size curve. This material is non-plastic, permeable, and was encountered in a low to high density condition. It will have no tendency to expand or consolidate. It will, however, have a tendency to settle upon application of foundation stresses or vibration. In the higher density states, it is not felt that settlement of this material will create any problems. Settlement could, however, be severe where shallow foundations are placed in low density zones of this material. Where this material was encountered at shallow foundation depth in higher density states, such as in Test Boring 1, 2, and 4, it should have a bearing capacity of at least 3000 psf. The bearing capacity may be considerably higher at certain locations. Since this material is nonexpansive no minimum pressure is required.

Soil Type No. [illegible] was found to be relatively free of sulfates.

Soil Type No. 3 classified as silt (ML) with a very large percentage of sand-sized particles, with only a slight change in grain-size characteristics. This material would have classified as silty sand. Generally, this material is nonplastic, of low to moderate permeability, and was encountered in a low density, high moisture condition. It will have no tendency to expand upon the addition of moisture, nor any tendency to true long-term consolidation under load. It will, however, exhibit considerable settlement upon application of foundation stresses and will be very low in bearing capacity. This material was encountered in a very limited portion of the soil profile, in Test Boring No. 5 between 5 and 10 feet. Where this material is encountered beneath structures, it is recommended that some type of deep foundation system be used, which penetrates the low density materials of Soil Type No. 3 and rests in the underlying gravel and cobbles. Soil Type No. 3 contains sulfates in detrimental quantities.

The moisture conditions observed in our test borings and the proximity of the Colorado River would indicate a strong potential for ground water and subsurface seepage beneath this site, particularly during wetter seasons. Ground water will probably be encountered in installation of drilled pier deep foundation systems, necessitating the use of casing and dewatering techniques during construction. Additionally, ground water may be encountered in deeper excavations for foundations for utilities.

Since the magnitude and nature of the proposed foundation loads are not precisely known to the Laboratory at this time the recommendations contained herein must be quite general in nature. Any special loads or unusual design conditions should be reported to the Laboratory so that changes in recommendations may be made if necessary. However, based upon our analysis of the soil conditions and project characteristics previously outlined, the following recommendations are made.

It is recommended that foundation systems constructed in this development penetrate any low density surficial materials and rest in the dense granular materials encountered in the second layer of the soil profile. In many instances shallow foundation systems would be suitable for the proposed structures. Shallow foundations typically would consist of continuous footings beneath bearing walls and isolated spread footings beneath columns and other points of concentrated load. The maximum allowable bearing capacity for the higher density coarse grained materials beneath this site will be variable depending upon soil conditions at any particular location. Areas where shallow foundation systems will be suitable are demonstrated by Test Borings 1, 2, and 4. By inspection of the enclosed Test Boring Location Diagram, it can be seen that these borings are primarily in the north and west portions of the subdivision. The bearing capacity of the denser alluvial materials encountered in these test borings should be at least 3000 psf and may be considerably greater in some locations. No minimum deadload pressure is required for these materials. The bottoms of foundations should be located a minimum of 2 feet below finished grade or greater if dictated by local building codes, for frost protection.

It is recommended that shallow foundation systems be well balanced. Foundation systems should be designed in such a manner that contact stresses are approximately the same at all points. This can be accomplished by placing larger footings beneath heavier loads and smaller footings beneath lighter loads. The balancing will depend somewhat upon the nature of the structure. Single-story slab on grade structures should be balanced on the basis of deadload only. Multi-story structures should be balanced on the basis of deadload plus approximately one-third the liveload. Using whichever criteria is applicable, the contact stresses beneath exterior foundation walls should be balanced to within $\pm$ 500 psf at all points. Isolated interior footings should be designed for unit loads of about 200 psf less than the average of those selected for the exterior walls.

All stemwalls for continuous shallow foundation systems should be designed as grade beams capable of spanning at least 12 feet. Horizontal reinforcing should be placed continuously around the structure with no gaps or breaks in the reinforcing steel, unless specially designed. Foundation walls should be reinforced at both top and bottom with the majority of the reinforcement being located at the bottom of the wall. Where foundation walls will retain soil in excess of 4 feet in height, vertical reinforcing may be necessary and should be designed. To design such vertical reinforcing the equivalent fluid pressure of the soil may be taken as about 40 pcf in the active state.

In some instances the upper portion of the coarser grain materials, as well as the overlying fine uation is evidenced in Test Borings, 3, 5, and 6 where competent bearing strata was not encountered until depths of 10 to 15 feet were achieved. As can be seen from inspection of the enclosed Test Boring Location Diagram, these borings were primarily on the east and southern portion of the site. In these instances, a deep foundation system consisting of driven piles or drilled piers will probably be most suitable. There are several advantages and disadvantages associated with each type of deep foundation system with respect to this site. Drilled pier and driven pile deep foundation systems will be discussed in turn.

If drilled piers are used on this site they should extend through any low density overlying materials and at least a minimum of 5 feet into the denser portion of the underlying alluvium. With this amount of penetration, the maximum and bearing capacity for piers may be taken as about 8000 psf. An allowable side friction of 1000 psf may be taken for the denser alluvial materials. Due to the low density condition of overlying materials, no additional contribution due to side friction for these materials should be considered in the design. There will be no minimum end bearing or side friction requirements for the alluvial gravels. It should be noted that difficulty may be encountered in the installation of drilled piers, due to the wet, soft nature of the overlying silts and the low density condition of the upper portion of the coarser grained materials in many locations. Also ground water will probably be encountered in drilled pier installation. Problems associated with loose caving soils and ground water problems will probably require the use of casing and dewatering techniques during construction.

The bottoms of all drilled piers should be thoroughly cleaned prior to placement of concrete. Piers should be reinforced continuously throughout their entire length. The amount of reinforcing required in each pier will depend upon the magnitude and nature of loads involved. As a rule of thumb, a minimum of one #5 rebar for every 16 inches pier circumference should be used with an absolute minimum requirement of two #5 rebars per pier.

To insure that all voids in the side walls are filled, concrete with a slump of 5 to 6 inches should be used. Piers having an extremely small diameter, on the order of 12 inches or less, may use concrete with a slump in excess of 6 inches. Piers must be thoroughly dewatered prior to the placement of concrete. If dewatering is not possible concrete should be tremmied below standing water. A freefall of concrete in excess of 5 feet should be prohibited unless the pier diameter is large enough to insure that concrete will not contact the side walls during the fall. Any casing used during drilling should be pulled as concrete is being placed to allow for the complete filling of all voids in the side walls with concrete.

The use of driven piles would eliminate the need for concern with respect to casing, caving soils and ground water problems. However, the capacity of a pile is much more difficult to establish during the design phase of a project than that of a drilled pier. Additionally, pile driving equipment may be less readily available in this area than the equipment used to install drilled piers. Therefore, the decision as to which type of deep foundation system is most suitable is purely economic and should be investigated by the owner or his representative.

Should it be decided to use driven piles a number of different pile types would be available for use. Piles typically consist of either timber, steel, or precast concrete. Each type of pile is associated with a number of advantages and disadvantages. Timber piles are typically suitable for design loads on the order of 10 to 50 tons which would be acceptable for many of the structures on this project. However, timber piles are often difficult to splice during driving and may be vulnerable to decay, should ground water level be subject to frequent fluctuation. Timber piles are comparatively low in cost and the problem with decomposition may largely be overcome by treatment of the pile.

Steel piles are very easy to splice making them suitable to sites where the bearing surface may fluctuate widely in depth. They are somewhat vulnerable to corrosion, however, particluarly in areas where the ground water may be rich in sulfates. The finer grain portions of the soils on this site can be expected to contain a signficant amount of sulfates. Steel piles are typically suitable for design loads on the order of 40 to 120 tons which would be more than sufficient for most of the structures in this subdivision.

Precast concrete piles are suitable for a very wide range of design loads. They can also be made to achieve a high corrosion resistance by the use of sulfate resistant cement in the concrete. However, they are typically associated with a fairly high initial cost and are often quite difficult to splice.

Specific recommendations pertaining to pile type and pile capacity cannot be easily made in a report of this nature, as a such a choice depends upon the expected loads, the driving equipment to be used, and other factors. Therefore, analysis of pile type and pile capacity will be left to the structural engineer. By way of example, however, a 12 inch diameter pile section which is driven to a resistance of 10 blows per inch, using a driving hammer in good repair with a rated energy of 15,000 foot pounds should be capable of developing a capacity somewhere between 25 and 40 tons. These estimated pile capacities are based on static considerations of bearing capacity and friction and may not precisely represent the true capacity of the pile obtained in the field. Therefore, when pile driving operations commence, pile capacities should be verified either by means of load tests or by use of a pile driving equation. Typical pile driving equations which are commonly used, would include the Modified Engineering News Record Equation and the Wave Equation.

Piles should be used in groups to provide for eccentricities in loading. The group capacity will be less than the summation of the individual pile capacities depending upon the relative spacing of piles. A conservative estimate of the group capacity, however, would be on the order of two-thirds of the summation of the individual pile capacities. Maximum spacing of piles should be twice the average pile diameter or 1.75 times the diagonal dimension of the pile cross section, but no less than 24 inches.

If horizontal loads exist and exceed 1000 pounds per pile, batter piles will be required. Hammer and cushioning should be matched to the chosen pile type, to assure the attainment of design load capacities during driving. No pile should be shorter than 10 feet in length. Vertical piles should not vary more than 2% from the plumb position. Eccentricity of reaction on a pile group with respect to the load resultant should not exceed a dimension that would produce overloads of more than 10% in any one pile.

Adequate drainage must be provided in the building area both during and after construction to prevent the ponding of water. The ground surface around the structures should be graded such that surface water will be carried quickly away. Minimum gradient within 10 feet of any structure will depend upon surface landscaping. Bare or paved areas should have a minimum gradient of 2%, while landscaped areas should have a minimum gradient of 5%. Roof drains must be carried across all backfilled areas and discharged well away from the structure. The overall drainage pattern should be such that water diverted away from one structure is not directed against an adjacent structure.

Floor slabs should be constructed in such a manner that they act independently of columns and bearing walls. Additionally, concrete floor slabs should be placed in sections no greater than 25 feet on a side. Deep contraction or construction joints could be placed at these lines to facilitate even breakage. This will help to keep to a minimum any unsightly cracking which would be caused by differential movement.

Floor slabs should be constructed over a capillary break or gravel bed of 4 to 6 inches in thickness. This gravel layer should not contain a significant amount of fines and should be provided with a free drainage outlet to the surface, so as not to act as a water trap beneath the floor slab. A vapor barrier is recommended beneath all floor slabs, placed directly above the capillary break layer.

Where considerable loads are placed on floor slabs, such as in the case of warehouse storage, special measures in floor slab construction may be necessary. These measures would include removal of the underlying low density materials and replaced with a suitable compacted fill, or the use of a structural floor system to prevent loads from being applied to the soft silt materials. Requirements for floor slab construction should be evaluated individually for each structure.

Backfill around the proposed structures and in utility trenches leading to the structures should be compacted to at least 90% of the maximum standard Proctor dry density, ASTM D-698. The native soils on this site may be used for backfilling purposes. Backfill should be placed in lifts not to exceed 6 inches compacted thickness and at a moisture content approximately equal to the Proctor optimum moisture content $\pm$ 2%. Backfill should be compacted to the required density by mechanical means. No water flooding techniques of any type should be used in the placement of fill on this site.

Any topsoil or debris should be removed from the constuction area prior to beginning of construction of foundations. Additionally, should any pockets of debris, low density materials, or otherwise unsuitable materials be encountered during excavation for footings this material should be removed and replaced with a suitable backfill compacted to at least 95% of the maximum standard Proctor dry density, using the procedures previously outlined.

Due to the nature of the structures to be constructed in this development and to the variable nature of the soil materials encountered, this report should be considered as preliminary site study. A more complete soils investigation is recommended for each structure prior to construction. This investigation would normally consist of additional auger borings. In this manner soil conditions beneath any given structure may be more precisely defined and the proposed project characteristics and construction techniques can be taken into account in foundation recommendations.

The finer grained soils on this site contain sulfates in detrimental quantities. Therefore, a sulfate resistant cement such as Type II Cement should be used in all concrete which will be in contact with the foundation soils. Under no circumstances should calcium chloride ever be added to a Type II Cement. In the event that a Type II Cement is difficult to obtain a Type I Cement may be used providing the concrete is separated from the soils by water resistant membranes.

It is believed that all pertinent points concerning the subsurface soils on this site have been covered in this report. If soil types and conditions other than those outlined herein are noted during construction, these should be reported to Lincoln-DeVore so that changes in recommendations may be made if necessary. Should questions arise or further information be required, please feel free to contact our office.

Lincoln DeVore
1000 West Fillmore St.
Colorado Springs, Colorado 80907
(303) 632-3593
Home Office

May 31, 1979

Excalibur Enterprises, Inc.
P O Box 2266
Grand Junction, CO 81501

Re: Hveem-Carmany Test
SIX & FIFTY WEST SUBD.

Gentlemen:

Personnel of Lincoln-DeVore have completed Hveem-Carmany testing on samples of material from the above referenced site. The results are as follows:

R = less than 5
Av. Displacement = 5.67 (@ 300 psi)
Av. Expansion Pressure = 0 (@ 300 psi)

It should be noted the material is unstable unless confined.

Respectfully submitted,

LINCOLN-DeVORE TESTING LAB., INC.

By George D. Morris, P. E.

/sam

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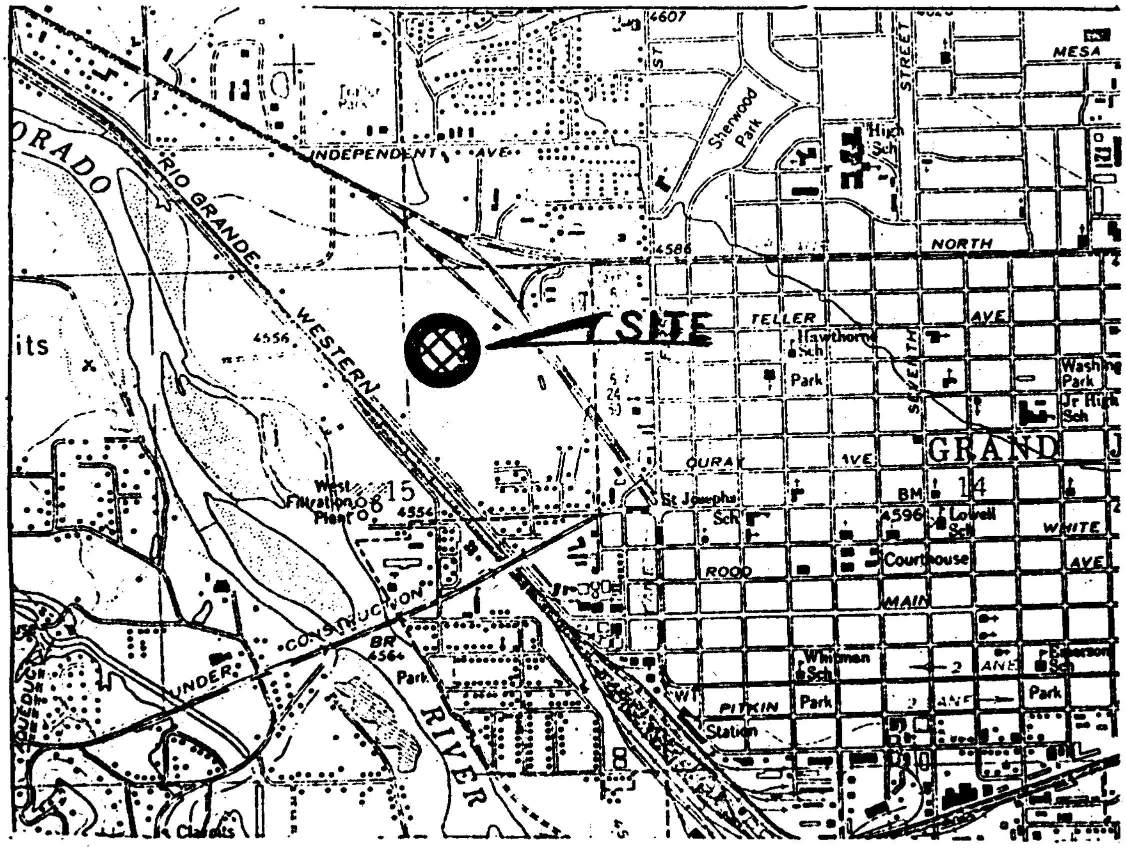


SCALE
1"=2000'

ADAPTED FROM
U.S.G.S. 7½' Quadrangles

SITE LOCATION MAP	THE LINCOLN-DeVORE TESTING LABORATORY COLORADO: Colorado Springs, Pueblo, Glenwood Springs, Montrose, Gunnison. WYOMING: Rock Springs

TEST HOLE No. / TOP ELEVATION

Test Hole No. 1

Silt, Fine Grain, Soft, Moist

GP Poorly Graded Gravel, Dense, Numerous Cobbles

ST Wc=17.0 (1)

50/12 Wc=12.6 (2)

50/6 Wc=8.7 (2)

Test Hole No. 2

ML Silt, Finegrain, Soft, Moist

GP Poorly Graded Gravel, Dense, Moist, Numerous Cobbles & Boulders.

Refusal at 15' Depth

ST Wc=19.6 (2)

46/12 Wc=9.0 (2)

50/0 Wc=8.8 (2)

Test Hole No. 3

ML Silt, Finegrain, Soft, Moist

GP Poorly Graded Gravel, Low Density, Moist, Numerous Cobbles & Boulders

Refusal at 13.5' Depth

46/12 Wc=14.9 (2)

11/12 Wc=10.0 (2)

5/12 Wc=16.9 (2)

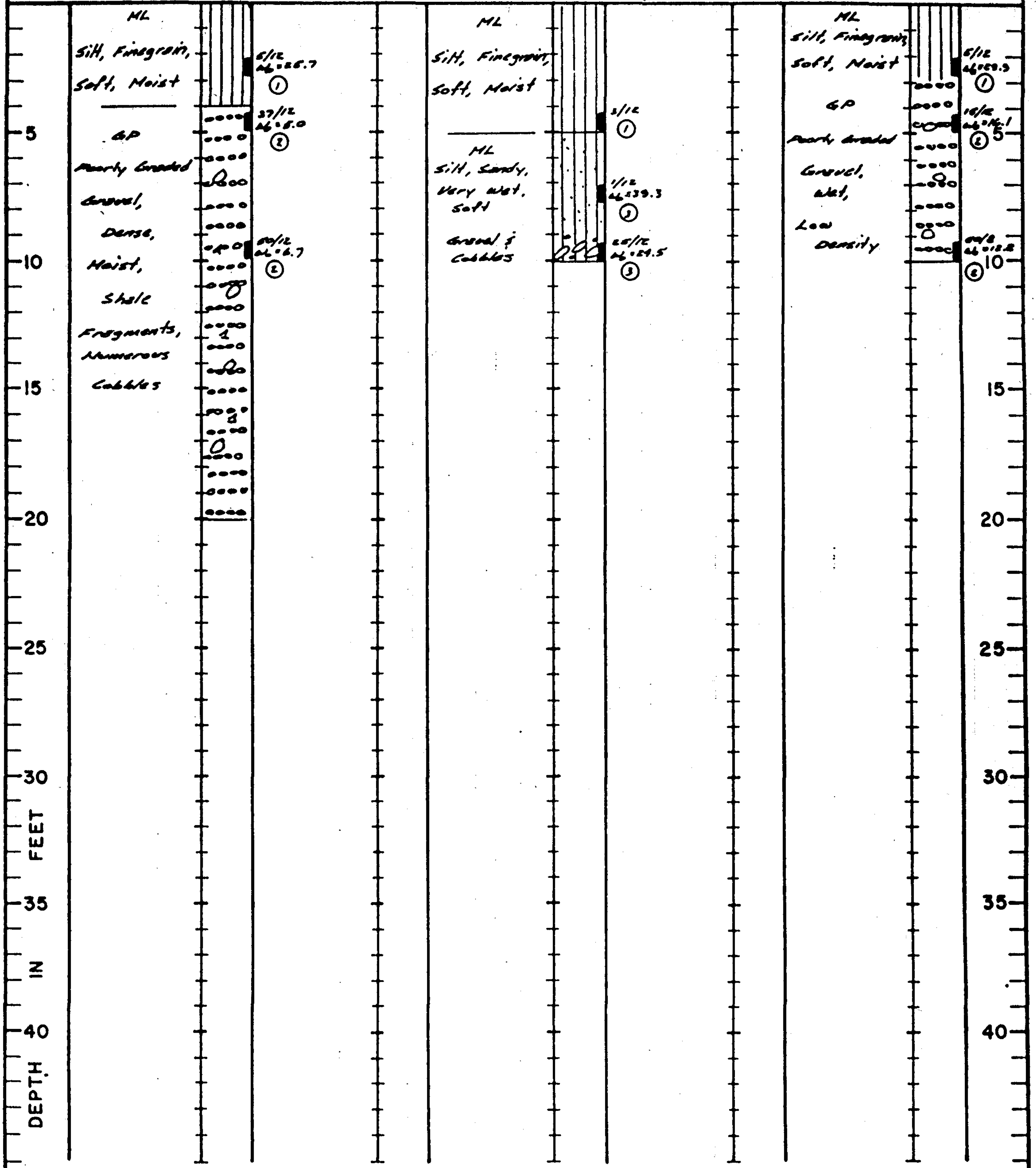
DEPTH IN FEET: 5, 10, 15, 20, 25, 30, 35, 40

DRILLING LOGS

LINCOLN DeVORE ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, PUEBLO, GLENWOOD SPRINGS, GRAND JUNCTION, MONTROSE, WYOMING: ROCK SPRINGS

TEST HOLE No.	4	5	6
TOP ELEVATION			



DRILLING LOGS

LINCOLN DeVORE
ENGINEERS · GEOLOGISTS

COLORADO: COLORADO SPRINGS, PUEBLO, GLENWOOD SPRINGS, GRAND JUNCTION, MONTROSE, WYOMING: ROCK SPRINGS

SUMMARY SHEET

Soil Sample SiH (ML)

Test No. J-381

Location 6th 50 West Subdivision Filing 3

D. 4/30/79

Boring No. 1 Depth 2'

Sample No. 1

Test by ZDT

Natural Water Content (w) 12.0 %

Specific Gravity (Gs) ______

In Place Density (γ_o) ______ pcf

SIEVE ANALYSIS:

Sieve No.	% Passing
1 1/2"	
1"	
3/4"	
1/2"	100
4	97.8
10	97.0
20	96.2
40	91.9
100	84.7
200	71.9

HYDROMETER ANALYSIS:

Grain size (mm)	%
.02	22.1
.005	9.2

Plastic Limit P.L. 20.0 %

Liquid Limit L. L. 20.0 %

Plasticity Index P.I. NP %

Shrinkage Limit ______ %

Flow Index ______

Shrinkage Ratio ______ %

Volumetric Change ______ %

Lineal Shrinkage ______ %

MOISTURE DENSITY: ASTM METHOD

Optimum Moisture Content - wo 12.8 %

Maximum Dry Density - γd 105.7 pcf

California Bearing Ratio (av) ______ %

Swell: ______ Days ______ %

Swell against ______ psf Wo gain ______ %

BEARING:

Housel Penetrometer (av) ______ psf

Unconfined Compression (qu) ______ psf

Plate Bearing: ______ psf

Inches Settlement ______

Consolidation % under psf

PERMEABILITY:

K (at 20°C) ______

Void Ratio ______

Sulfates 1000+ ppm.

SOIL ANALYSIS

LINCOLN-DeVORE TESTING LABORATORY
COLORADO SPRINGS, COLORADO

Soil Sample *Poorly Graded Gravel (GP)*

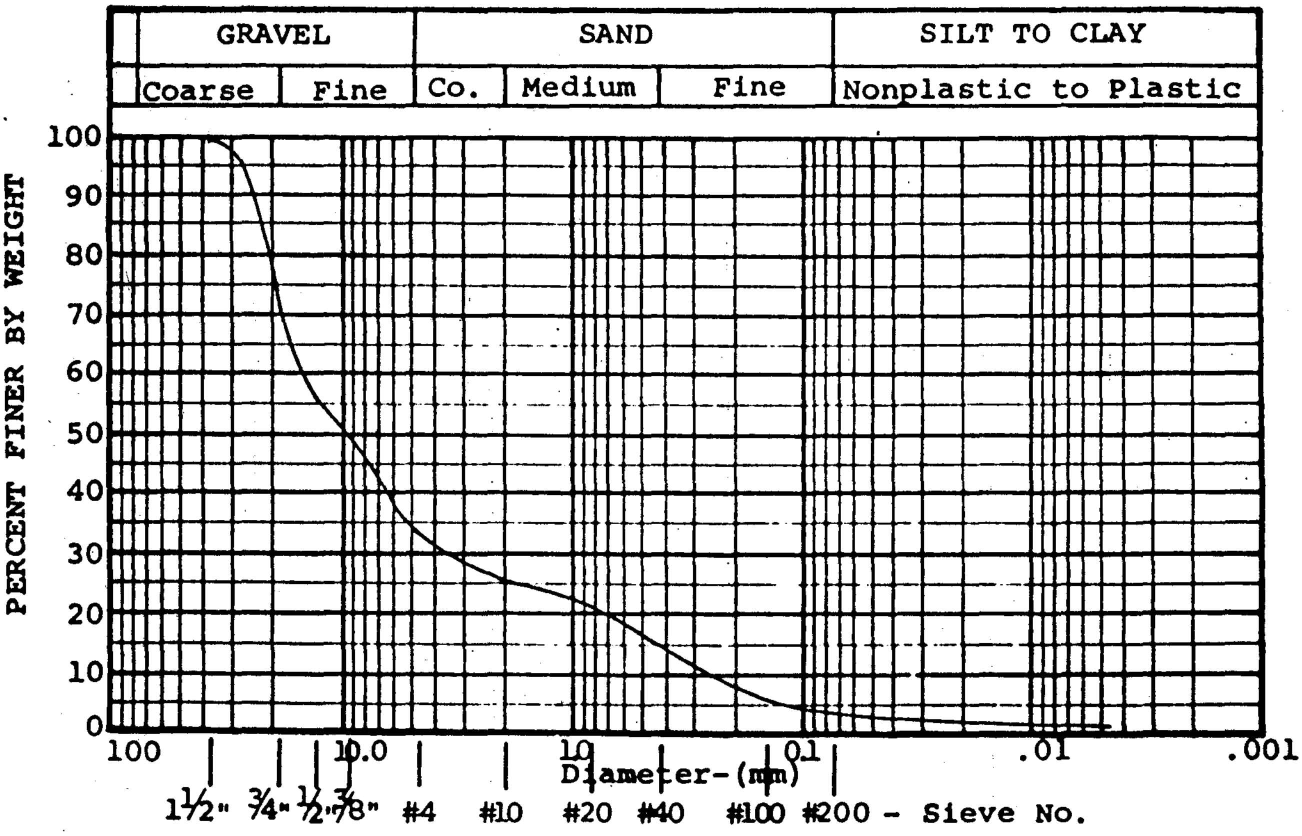
Test No. *V-381*

Project *6 & 50 West Subd. Filing 3*

Date *4/30/79*

Sample Location *TH-2, 5' Depth*

Test by *ZDT*



Sample No.	*2*
Specific Gravity	
Moisture Content	*9.0*
Effective Size	*0.24*
Cu	*54*
Cc	*3.5*
Fineness Modulus	

L.L. ____ % P.I. *NP* %

BEARING *3000* psf

Sieve Size	% Passing
1 1/2"	*100*
1"	*95.2*
3/4"	*74.3*
1/2"	*56.1*
3/8"	*48.7*
4	*33.5*
10	*25.0*
20	*21.6*
40	*15.0*
100	*5.5*
200	*3.7*
.0200	*1.5*
.005	*0.9*

Sulfates *Negative* ppm

GRAIN SIZE ANALYSIS

LINCOLN-DeVORE TESTING LABORATORY
COLORADO SPRINGS, COLORADO

SUMMARY SHEET

Soil Sample *Sandy silt (ML)*

Test No. *J-381*

Location *6 & 50 West Subdivision Filing 3*

Date *5/10/79*

Boring No. *5* Depth *7'*

Sample No. *3*

Test by *TDH*

Natural Water Content (w)_____%

Specific Gravity (Gs) *2.70*

In place Density (γ_o)_____pcf

SIEVE ANALYSIS:

Sieve No.	% Passing
1 1/2"	
1"	
3/4"	
1/2"	100
4	94.3
10	86.3
20	82.4
40	74.8
100	67.6
200	51.0

HYDROMETER ANALYSIS:

Grain size (mm)	%
.02	18.0
.005	7.5

Plastic Limit P.L._____%

Liquid Limit L. L._____%

Plasticity Index P.I. *Nonplastic* %

Shrinkage Limit *21.7* %

Flow Index_____

Shrinkage Ratio_____%

Volumetric Change_____%

Lineal Shrinkage_____%

MOISTURE DENSITY: ASTM METHOD

Optimum Moisture Content - w_o_____%

Maximum Dry Density - γ_d_____pcf

California Bearing Ratio (av)_____%

Swell:_____Days_____%

Swell against_____psf Wo gain_____%

BEARING:

Housel Penetrometer (av)_____psf

Unconfined Compression (qu)_____psf

Plate Bearing:_____psf

Inches Settlement_____

Consolidation % under psf

PERMEABILITY:

K (at 20°C)_____

Void Ratio_____

Sulfates *500+* ppm.

SOIL ANALYSIS

LINCOLN-DeVORE TESTING LABORATORY
COLORADO SPRINGS, COLORADO

Soil Type surface sample

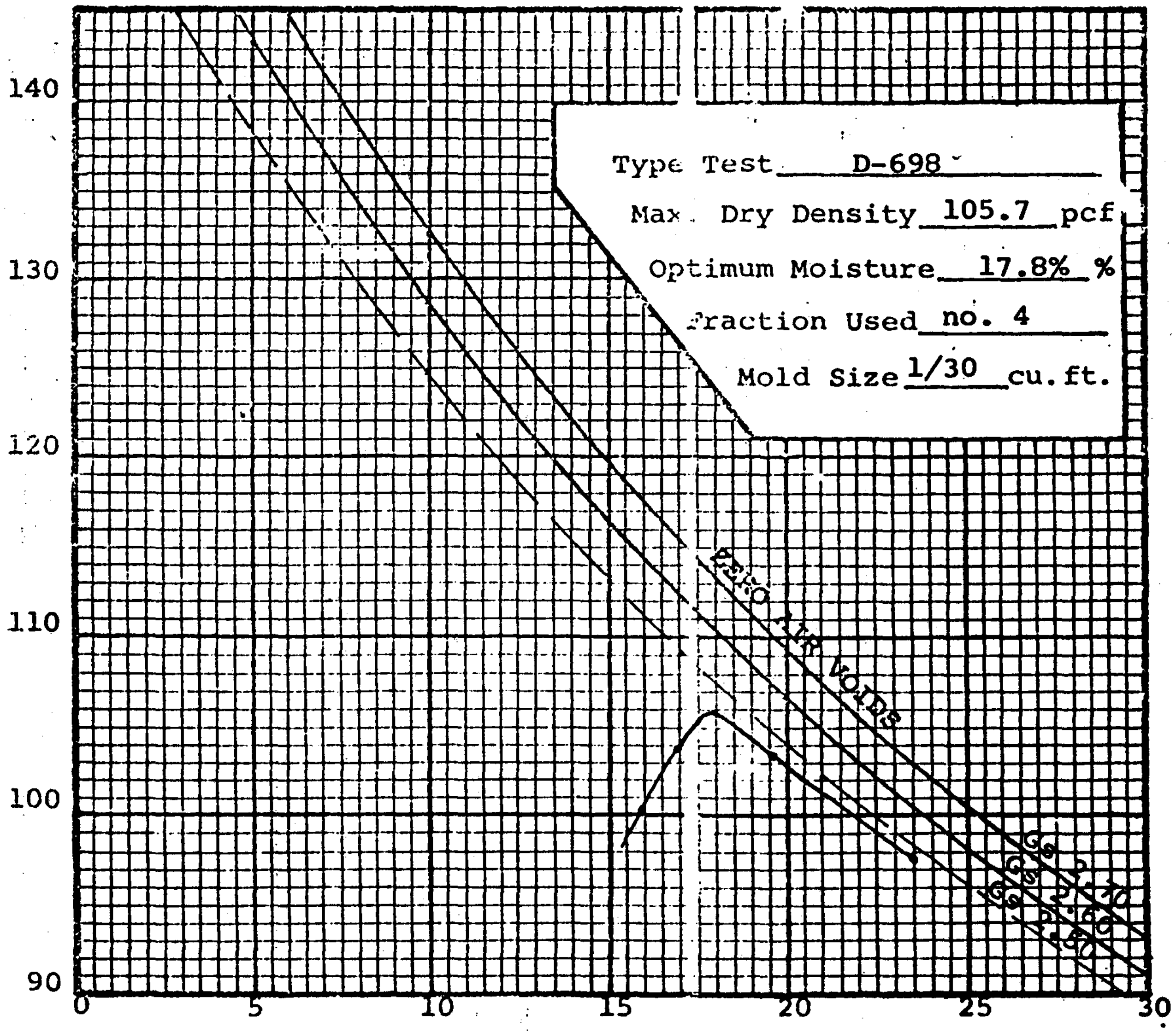
Project 6&50 Subdivision West

Sample Obtained from surface

Test No. D-698

Date 3-23-79

Test by SJC



SOIL PROPERTIES

Specific Gravity ______	In Situ Moisture Content ______ %
Unified Classification ______	In Situ Density (av.) ______ pcf
Liquid Limit ______	Field CBR value ______ %
Plastic Limit ______	Laboratory CBR value ______ %
Plasticity Index ______	Three day swell ______ %

MOISTURE-DENSITY RELATION

LINCOLN-DeVORE TESTING LABORATORY
COLORADO SPRINGS, COLORADO

J-381

REVIEW SHEET SUMMARY

FILE NO. 49-82 TITLE HEADING 6 & 50 West Subdivision DUE DATE 7/12/82

ACTIVITY - PETITIONER - LOCATION - PHASE - ACRES Petitioner: Albino Venegas. Location: West side of Highway 6 & 50, South of North Avenue line, East of 25.5 Road. A request for a final plat on approximately 18 acres in a light commercial zone. Consideration of final plat.

PETITIONER ADDRESS Box 1883 Grand Junction.

ENGINEER Paragon Engineering

DATE REC.	AGENCY	COMMENTS
7/8/82	City Utilities	It is stated that water will be provided by Ute Water. I don't think Ute has a water system in the area. The storm drainage system for previous filings was never built. The City Engineer should confirm that the storm drainage plan shown is compatible to the previous filings.
7/9/82	Planning Staff Comments	1. Resolve all previous concerns re: drainage, highway access, development to the north and south. NOTE: This is straight zone so all development will have to meet the reuqirements of that zone at the time of building permit. 2. This department has no problem if all other review agency concerns resolved.
7/9/82	Trans. Engineer	The traffic circulation looks OK. I assume the Highway Department is in agreement with the frontage road access.
7/12/82	City Engineer	Ute doesn't have lines in this area. Water ties shown are to City lines. In light of the ongoing dilemma concerning storm drainage outletting in this area, I recommend the City Council consider ordering a street (and storm drain) improvement district for early 1983 to build 25 1/2 Road and the storm drain adjacent to 6 & 50 Filing 2, 6 & 50 Filing 3 and North Avenue West Commercial Subdivision. Assessments for these improvements should be for full-cost and should be addressed against all the lots in the proposed subdivisions on some equitable basis rather than to have the adjacent frontage pay for the entire assessment. Access and storm drainage deficiencies in this area are very legitimate issues and since the property owners do not seem able to resolve these land development responsibilities, the City should. If an improvement district is ordered, the City Engineer will design the storm drain outlet system along with 25 1/2 Road. If not, this petitioner should construct a storm drainage outlet system in accordance with the previously approved plans prepared by Paragon Engineering. Colorado Division of Highways approval should be obtained for the frontage road access and improvements. Attached is a copy of the previously CDH-approved plan. (Note the approved plan shows an intersection with right of way dedication which is different from this current proposal.) Sanitary sewer system is not acceptable since sewers are shown at less than State Health Department and city minimum grades.
7/12/82	State Highway	Per conversation with State Highway Department, Ed Gebhardt, the petitioners proposal needs to be coordinated with the State Highway's overall plan regarding the frontage road and highway access. This needs to be approved by the State Highway prior to any construction being started. A State Highway approval will be necessary to release the petitioner and ensure overall plan as previously approved.
7/13/82	City Fire Late	
7/15/82	P.S. Co. - Late St. Hwy Dept.	

DATE REC.	AGENCY	COMMENTS
8/5/82	GJPC Minutes of 7/27/82	MOTION: (COMMISSIONER O'DWYER) "ON ITEM #49-82, 6&50 WEST SUBDIVISION, FILING #3 -- FINAL PLAT, I MOVE WE FORWARD THIS TO CITY COUNCIL WITH THE RECOMMENDATION FOR APPROVAL BASED UPON THE RESOLVING THE DRAINAGE PROBLEM BEFORE ANYTHING CAN BE DONE AND ALL OTHER STAFF CONCERNS." COMMISSIONER DUNIVENT SECONDED THE MOTION. CHAIRWOMAN QUIMBY REPEATED THE MOTION, CALLED FOR A VOTE, AND THE MOTION CARRIED UNANIMOUSLY.

July 22, 1982

RESPONSE TO REVIEW SHEET COMMENTS

RECEIVED MESA COUNTY
DEVELOPMENT DEPARTMENT
JUL 22 1982

File No: 49-82
Item: 6 & 50 West Subdivision
Filing No. Three
Phase: Final Plat
Location: West of Hwy 6 & 50, South of North Ave. Line

Agency	Response
City Utilities	1. Water service will be by the City of Grand Junction. 2. In 1978 a system for disposal of storm water was designed by this office, and approved for construction by the City Engineer for 6 & 50 Subdivision, Filing No. Two. It was never installed. Construction of this system would solve the drainage problem that currently exists in this area. The developer of 6 & 50, Filing Three is willing to pay, escrow, or give Power of Attorney to construct the system. Since several subdivisions would benefit, it would be fair that they all pay for it. This is being discussed between the developers. As an alternative, the City could order a full-cost improvement district to design and construct 25½ Road, including a storm drain, and solve the drainage problem. This would likely prove more costly to developers in the area.
Transportation Engineer	Commented that the circulation pattern was acceptable, and access is to the frontage road as noted in State Highway response below.
City Engineers	1. Water service will be by the City of Grand Junction. 2. Please refer to City Utilities response (2) above. 3. Please refer to State Highway response below. 4. The sanitary sewer mains are to be 10", laid at a 0.25% (minimum) grade.
State Highway	The developer of 6 & 50 West, Filing Three proposes to construct the frontage road as shown in the plans. This is a temporary road. Ultimately, West Hill Ave. will access directly to Hwy 6 & 50. The frontage road will then be relocated across Tracts A and B (on plat).
City Fire	Indicated no objections to hydrant placement. Hydrants and mains shall be installed prior to street paving or building occupancy.
City Staff	1. Please refer to City Utilities response. (2) regarding drainage and participation with adjacent developments. 2. Please refer to State Highway response regarding access. 3. Building developments shall meet the requirements of the zone.

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Acres 18+

Units ______

Density ______

final plat

Activity 6&50 West Subdivision-Filir

Phase ______

Common Location W side of 6&50, S. of

Date Submitted 7/1/82 Date Mailed Out 7/2/82

10 day Review Period Return by 7/12/82

Date Adjacent Property Owners Notified of MCPC/GJPC ______ (not previously submitted) Date Adjac

review agencies

review agencies	A	B	C	D	E	F	G	H	I	J	K	~~L~~	M	N	O	~~P~~
Development Dept.	●			●	●	●	●		●	●		●	●	●	●	●
County Road	●			●	●	●			●	●			●	●	●	
County Health	●			●	●	●			●			●	●			
County Surveyor	●			●	●	●										
County Parks/Recreation	●			●	●	●										●
County Engineer	●			●	●	●			●	●		●	●	●	●	
Transportation Engineer	●			●	●	●									●	
City Engineer	●			●	●	●			●	●		●	●	●	●	
City Utilities (2 sets)	●			●	●	●				●			●	●	●	
City Parks/Recreation	●			●	●	●										●
City Police Dept.	●			●	●	●										
County Sheriff	●			●	●	●										
Floodplain Administration	●			●	●	●						●				
Comprehensive Planning	●			●	●	●										
G.J. Dept. of Energy	●			●	●	●										
Fire City	●			●	●	●							●		●	
Irrigation	●			●	●	●							●	●	●	
Drainage	●			●	●	●							●	●	●	
Water (Ute, Clifton)	●			●	●	●							●		●	
Sewer	●			●	●	●							●	●	●	
G.V. Rural Power	●			●	●	●							●			
Mountain Bell	●			●	●	●							●			
Public Service (2 sets)	●			●	●	●							●			
Soil Conservation	●			●	●	●			●	●				●		
State Highway Dept.	●			●	●	●			●	●		●	●	●	●	
State Geological	●			●	●	●			●	●				●		
State Health Dept.	●			●	●	●			●			●				
Transamerica	●			●	●	●										
Water & Power Resources	●			●	●	●							●			
10 Mack, Mesa, Collbran, Palisade, Fruita, DeBeque, G.J., Mesa Cnty.	●			●	●	●										
17 OTHER: CC/PC (P)	●			~~●~~	●	~~●~~							●			
1 client	o			o	o	o	o		o	o			o	o	o	
totals	28			11	28	11	2		4	6			27	6	8	