Table of Contents

File 1990-0043 Name 1st Presbyterian Church – Revised Final Plan NE Corner of 27.5 & Cortland Avenue

	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	**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
X		Reduction of assessor's map.
		Evidence of title, deeds, easements
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 non-bound reports
		Traffic studies
		***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 – 11/6/90	X	X	Letter from Don Watkins, Reck and Associates re: Building specifications– 10/22/90
X	X	Development Summary – 1/6/90	X	X	Letter from Don Watkins, Reck Associates to Bill Cheney, Public Works re: revised final plan – 10/24/90
X	X	Review Sheet Summary – 11/6/90	X	X	Letter from Don Watkins, Reck Associates to George Bennett, Fire Dept. re: fire flow – 10/24/90
X		Review Sheets	X		Letter from Don Watkins, Reck and Associates to Jeff Wendland, Walker Field Airport re: aviagation easement – 10/24/90
X	X	Planning Clearance - ** - 12/12/91	X		Letter from Don Watkins to Gary Matthews, Ute Water District re: easement will not be provided for the fireline system – 10/24/90
X	X	Planning Commission Minutes - ** - 11/6/90	X	X X	Letter from Don Watkins to Don Newton, Public Works re: road improvements – 10/24/90
X		Public Notice Posting – 10/24/90	X	X	Letter from Mark D. Young, Rolland Eng. to Don Newton re: review comments pertaining to the drainage/grading plan – 11/1/90
X		Quit Claim Deed – Book 1873 / Page 162 - 1/2/92	X	X	Letter from Don Watkins to Dave Thornton re: comments and concerns of the agencies
X	X	Avigation Easement – Book 1859 / Page 975	X	X	Memo from Don Newton to Dave Thornton re: satisfactorily addressed comments – 11/690
X	X	Project Narrative	X	X	Letter from Ken Etter to Planning re: CUP issued in 1986 violated zoning criteria – 11/7/90

X		Bank Guarantee Agreement and Improvements Agreement - unsigned	X	X	Memo from John Shaver to Dan Wilson re: approval of final plan – church is in airport critical zone designation - 11/13/90
X		Warranty Deed – Book 1589 / Page 413 – 4/8/86	X	X	Letter from Dan Wilson to Kenneth Etter re: response to letter received 11/8/90
X		Development Application - 10/1/90	X		Memo from Kathy Portner to Neva Lockhart re: item would be pulled from workshop agenda – 11/20/90
		Sign Permit - ** -	X	X	Letter from Dave Thornton to Kenneth Etter re: response to 12/17/91 letter – 1/6/92
X		Promissory Note – Book 1876 / Page 338	X	X	Comments and Recommendations regarding the Major Arterial Corridor Study and the Northeast Area Transportation Study – 10/1/90
X		Certificate of Occupancy – 12/8/92 – also temporary effective until May 15, 1993	X	X	Drainage Report
X	X	Grading and Drainage Plan – 9/90 and Revised 9/91(in file)	X	X	Utility Easement - ** - copy to City Clerk
X	X	Elevation Plan	X	X	Geotechnical Engineering Study – 9/25/90
X	X	Site Plan	X		Planting Plan – 8/6/91
X	X	Landscape Submittal Plan	X		Electrical Plan – 10/18/91
X		Topographic Survey – 9/90	X		Notices of Public Hearing – 11/21/90
X	X	Improvements Agreement , Promissory Note, Release of Obligation Book 1876 / Page 338 – 345 – 1/24/92 & Book 2240 / Page 396 – 6/12/96 - **	X	X	Wet Tap Connections

Original
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#43 90

FIRST PRESBYTERIAN CHURCH REVISED FINAL PLAN

PROJECT NARRATIVE

The First Presbyterian Church of Grand Junction proposes to develop the property located at the Northeast corner of Cortland Avenue and 27 1/2 Road. The development will consist of a religious facility to be staged in two or more phases. The first phase will contain a sanctuary to seat 440 and supporting spaces (administration, education, fellowship/meeting hall, kitchen, choir practice room and other miscellaneous spaces). Ultimate build-out is anticipated to encompass a sanctuary to seat 750, a chapel to seat 65, supporting spaces listed above, a gymnasium and possible day-care usage.

Parking for phase one will be provided at one space for each three seats at the sanctuary (147 spaces total), ultimate build-out will provide one space for each three seats at the sanctuary (approximately 250 spaces total). Vehicular access will be maintained off of Cortland Avenue and 27 1/2 Road during all phases of development. Recreational facilities are anticipated to be developed on the site at some undetermined date. These might include courtyards, playgrounds, a softball field, tennis court and picnic area. The portions of the site not developed during the initial phase will be maintained in a natural state pending further development.

Drainage/storm sewer improvements will be implemented to accommodate new development and historic flows across this property from adjacent properties. These improvements will be phased with the development of the project to accommodate only the build-out at any given time. An existing drainage ditch across the south end of the site will be rerouted around or through the new parking improvements.

DEVELOPMENT SCHEDULE

The initial phase of development of the property at the Northeast corner of Cortland Avenue and 27 1/2 Road is anticipated to commence around the middle of March 1991. Construction would continue eight to ten months thereafter with completion October to December of 1991. Street improvements along Cortland Avenue would be completed in conjunction with the first phase. Development of subsequent phases of the building and site are undetermined at this time.

UTE WATER CONSERVANCY DISTRICT

POST OFFICE BOX 460

GRAND JUNCTION, COLORADO 81502

Attachment "A"

50-25 ROAD

TELEPHONE 242-7491

CURRENT RATES & CHARGES IN EFFECT........................JANUARY 1, 1985

WET TAP CONNECTIONS
(for system expansion or extension)

CONNECTION SIZE	Transmission or Distribution Main Size							
	24"	18"	16"	14"	12"	10"	8"	6"
12"	$4500	$3900	$3700	$3100	$2900			
10"	4100	3500	3300	2900	2500	$2100		
8"	3700	3100	3100	2600	2100	1900	$1600	
6"	3500	2900	2700	2300	1900	1700	1500	$1300
4"	3100	2700	2500	2000	1700	1500	1300	1000
	4" x 4" or smaller = $700.00							

TAP FEE (metered service)	WATER RATE (measured consumption)
3/4" meter $3,200.00	$8.00 minimum for 3,000 gal. over 3,000 gal. @ $2.10/1000.
1" meter $4,800.00	$8.00 minimum for 3,000 gal. over 3,000 gal. @ $2.10/1000.
1½" meter $7,200.00	$40.00 minimum for 15,000 gal. next 15,000 gal. @$2.10/1000, over 30,000 gal. @$2.00/1000.
2" meter $10,600.00	$64.00 minimum for 24,000 gal. next 24,000 gal. @$2.10/1000, over 48,000 gal. @$2.00/1000.
3" meter $19,200.00	$140.00 minimum for 52,500 gal. next 52,500 gal. @$2.10/1000, over 105,000 gal. @$2.00/1000.
4" meter $33,600.00	$240.00 minimum for 90,000 gal. next 90,000 gal. @$2.10/1000, over 180,000 gal. @$2.00/1000.
6" meter $84,000.00	$560.00 minimum for 210,000 gal. next 210,000 gal. @$2.10/1000, over 420,000 gal. @$2.00/1000.

Over 6" requires District Board approval.

FIRELINE DETECTOR CHECK VALVE
(protection of private lands and/or sprinkled buildings)

SIZE:	4" or less	6"	8"	10"	12"
TAP FEE:	$4800.00	$5600.00	$7500.00	$10,000.00	$14,000.00
RATE PER MONTH:	$ 25.00	$ 50.00	$ 75.00	$ 100.00	$ 125.00

All Fees include the apparatus and necessary related materials, equipment and labor for installation by UTE personnel.

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#43 90

Onion Hill Ltd.
P.O. Box 2185
Grand Junction, CO 81502

Andrew Christensen Family
Ltd. Partnership
2669 Paradise Drive
Grand Junction, CO 81506

Earl H. & Alice C. Davis
P.O. Box 2783
Grand Junction, CO 81502

Emanuel Epstein
1900 Quentin Road
Brooklyn, NY 11229

Jimmie L. Etter
697 27 1/2 Road
Grand Junction, CO 81501

Ptarmigan Estate
P.O. Box 60214
Grand Junction, CO 81506

William A. & Judith C. Ihrig
2324 N. Serville Cir.
Grand Junction, CO 81506

R. Butler & Karen S. Arnold
2202 Dogwood Court
Grand Junction, CO 81506

Betty June Schumann
3972 S. Piazza Place
Grand Junction, CO 81506

T.L. Benson
Susan K. Gazdak
2357 E. Piazza Place
Grand Junction, CO 81506

Paul E. & Marjorie A. Kemper
1111 Horizon Drive #305
Grand Junction, CO 81506

Thomas A. & Mary A. Foster
2298 N. Seville Cr.
Grand Junction, CO 81506

Henry I. & Virginia S.
Johnson Jr.
2285 S. Seville Circle
Grand Junction, CO 81506

William E. & Phyllis E.
Trainor
2297 Seville Circle
Grand Junction, CO 81506

Jerry Elliott
998 24 Road
Grand Junction, CO 81505

Harvey S. & Margaret L.
Huffer
2298 S. Seville Circle
Grand Junction, CO 81506

Warren & Magorie Bystedt
4406 Spring St.
Davenport, IA 52807

James F. & Dianna L. Pasqua
3969 S. Piazza Lane
Grand Junction, CO 81506

Wesley J. and Delores K.
Pidcock
2256 S. Seville Circle
Grand Junction, CO 81506

John H. & Muriel F. Crawford
3943 S. Piazza
Grand Junction, CO 81506

JoAnn M. Graham
1251 Bookcliff Apt. #11
Grand Junction, CO 81501

William D. & Christina C.
Potter
2297 N. Seville Circle
Grand Junction, CO 81506

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from Office

#43 90

ADJACENT PROPERTY NOTIFICATION

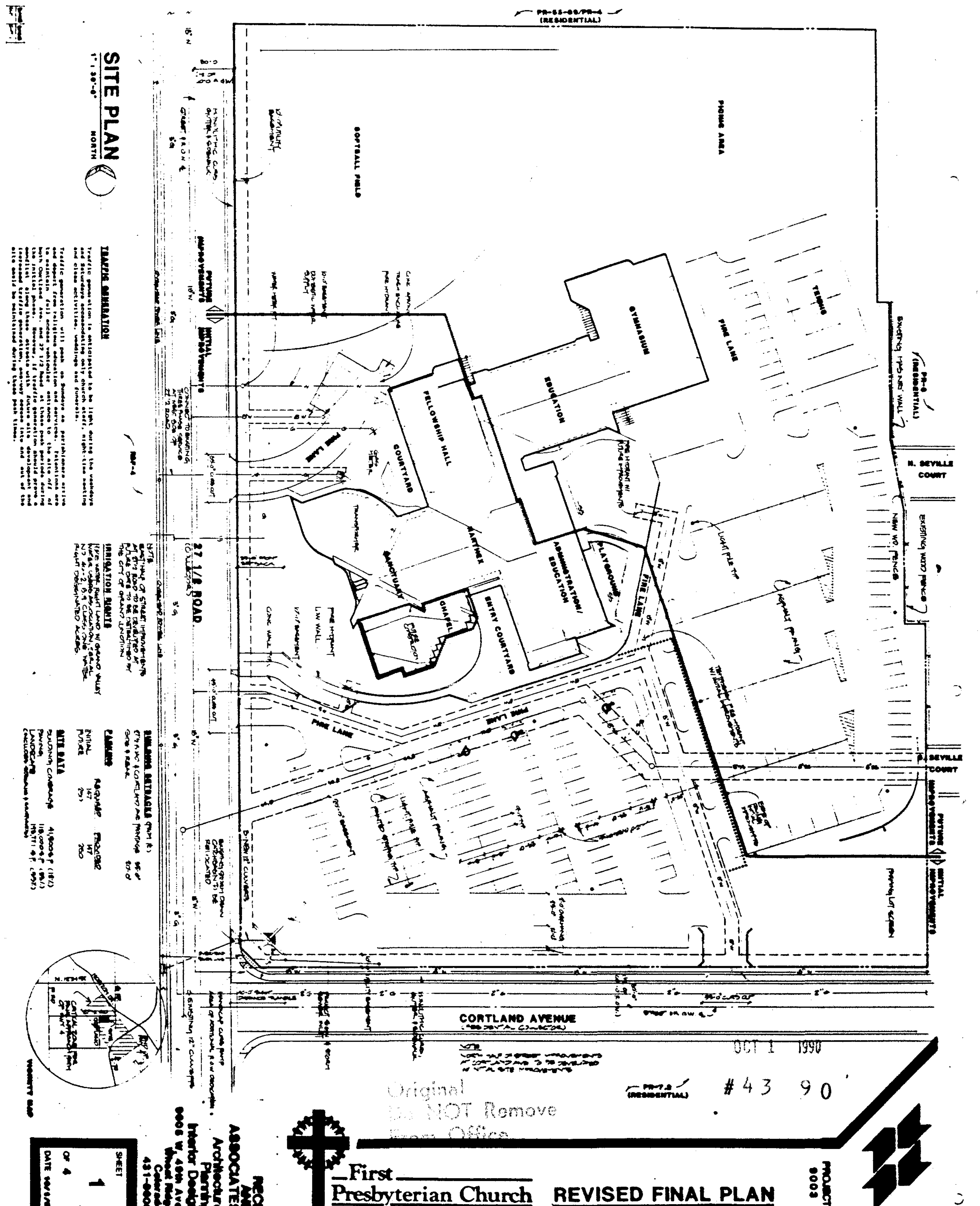
FIRST PRESBYTERIAN CHURCH
REVISED FINAL PLAN

Property	Owner
2945-011-00-035	Onion Hill Ltd. P.O. Box 2185 Grand Junction, CO 81502
2945-012-00-011	Earl H. & Alice C. Davis P.O. Box 2783 Grand Junction, CO 81502
2945-012-00-033	Andrew Christensen Family Ltd. Partnership 2669 Paradise Drive Grand Junction, CO 81506
2945-012-00-052	Earl H. & Alice C. Davis P.O. Box 2783 Grand Junction, CO 81502
2945-012-00-053	Earl H. & Alice C. Davis P.O. Box 2783 Grand Junction, CO 81502
2945-012-00-073	Emanuel Epstein 1900 Quentin Road Brooklyn, NY 11229
2945-012-00-074	Jimmie L. Etter 697 27 1/2 Road Grand Junction, CO 81501
2945-011-36-001	Henry I. & Virginia S. Johnson Jr. 2285 S. Seville Cr. Grand Junction, CO 81506
2945-011-36-002	William E. & Phyllis E. Trainor 2297 Seville Cr. Grand Junction, CO 81506
2945-011-36-016	Thomas A. & Mary A. Foster 2298 N. Seville Cr. Grand Junction, CO 81506

Property	Owner
2945-011-36-017	Paul E. & Margorie A. Kemper 1111 Horizon Drive #305 Grand Junction, CO 81506
2945-011-36-018	Betty June Schumann 3972 S. Piazza Place Grand Junction, CO 81056
2945-011-36-019	T.L. Benson Susan K. Gazdak 2357 E. Piazza Place Grand Junction, CO 81506
2945-011-37-001	Jerry Elliott 998 24 Road Grand Junction, CO 81505
2945-001-37-002	Harvey S. & Margaret L. Huffer 2298 S. Seville Circle Grand Junction, CO 81506
2945-011-37-009	William D. & Christina C. Potter 2297 N. Seville Circle Grand Junction, CO 81506
2945-011-37-010	Jerry Elliott 998 24 Road Grand Junction, CO 81505
2945-011-38-001	James F. & Dianna L. Pasqua 3969 S. Piazza Lane Grand Junction, CO 81506
2945-011-38-002	T.L. Benson Susan K. Gazdak 2357 E. Piazza Place Grand Junction, CO 81506
2945-011-39-001	Wesley J. & Delores K. Pidcock 2256 S. Seville Circle Grand Junction, CO 81506
2945-011-39-002	John H. & Muriel F. Crawford 3943 S. Piazza Grand Junction, CO 81506
2945-011-40-001	JoAnn M. Graham 1251 Bookcliff Apt. 11 Grand Junction, CO 81501

Property	Owner
2945-011-46-002	Ptarmigan Estate P.O. Box 60214 Grand Junction, CO 81506
2945-011-46-003	William A. & Judith C. Ihrig 2324 N. Serville Circle Grand Junction, CO 81506
2945-011-46-004	Ptarmigan Estate P.O. Box 60214 Grand Junction, CO 81506
2945-011-46-005	R. Butler & Karen S. Arnold 2202 Dogwood Court Grand Junction, CO 81506
2945-011-46-006	Ptarmigan Estate P.O. Box 60214 Grand Junction, CO 81506
2945-011-46-009	Ptarmigan Estate P.O. Box 60214 Grand Junction, CO 81506
2945-011-46-010	Ptarmigan Estate P.O. Box 60214 Grand Junction, CO 81506
2945-011-46-011	Ptarmigan Estate P.O. Box 60214 Grand Junction, CO 81506
2945-011-46-012	Ptarmigan Estate P.O. Box 60214 Grand Junction, CO 81506
2945-011-46-013	Ptarmigan Estate P.O. Box 60214 Grand Junction, CO 81506
2945-011-46-014	Ptarmigan Estate P.O. Box 60214 Grand Junction, CO 81506
2945-011-46-015	Ptarmigan Estate P.O. Box 60214 Grand Junction, CO 81506

Property	Owner
2945-011-46-016	Betty J. Schumann 3986 S. Piazza Grand Junction, CO 81506 ✓
2945-011-46-017	Warren and Marjorie Bystedt 4406 Spring St. Davvenport, IA 52807 ✓
2945-011-46-018	Betty J. Schumann 3986 S. Piazza Grand Junction, CO 81506 ✓



OCT 1 1990

#43 90

Original Do NOT Remove From Office

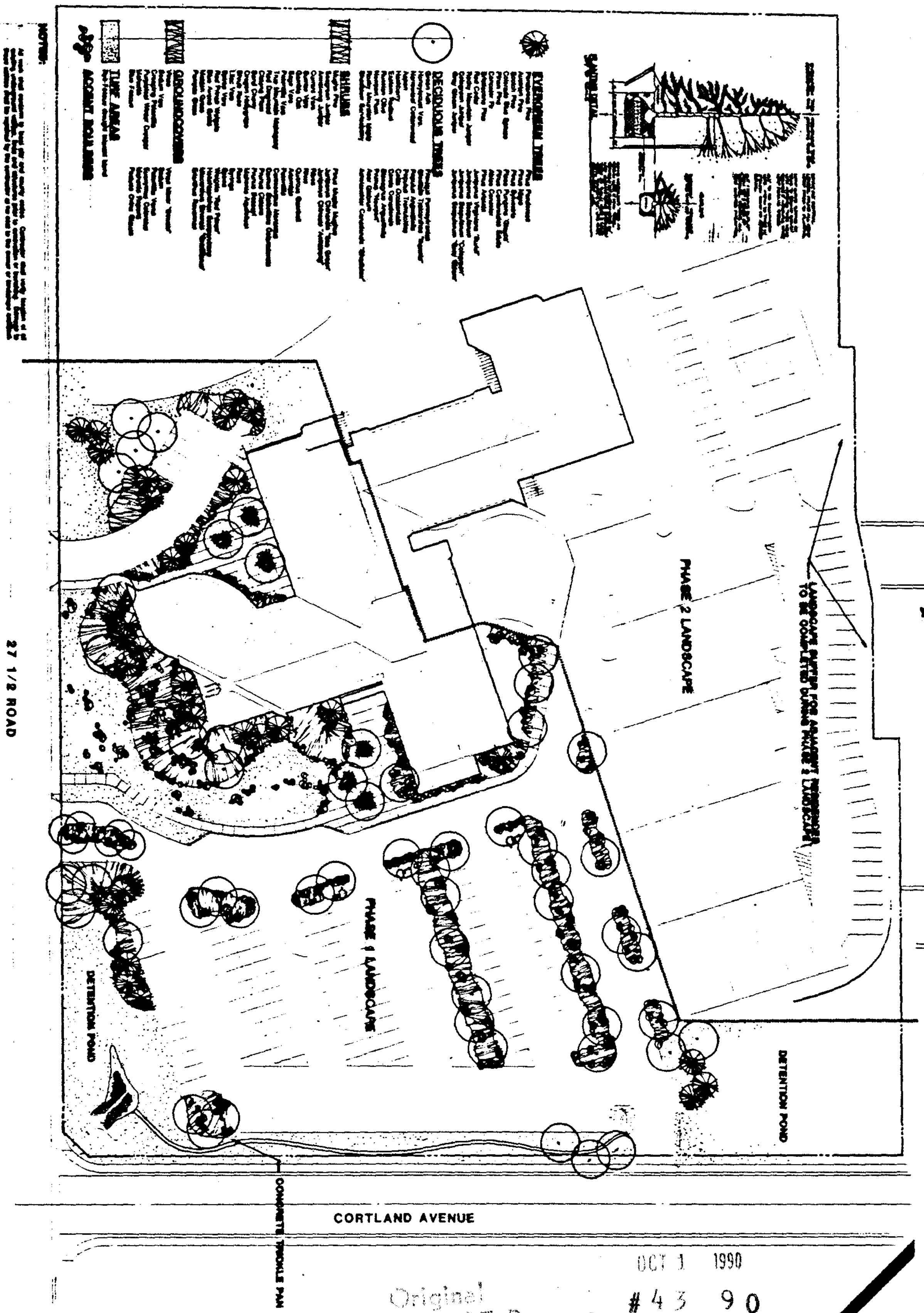
RECK AND ASSOCIATES
Architecture
Planning
Interior Design
9008 W. 49th Ave.
Wheat Ridge
Colorado
431-0600

SHEET 1 OF 4

First Presbyterian Church
GRAND JUNCTION, COLORADO

REVISED FINAL PLAN

PROJECT 9003



LANDSCAPE SUBMITTAL PLAN

OCT 1 1990

#43 90

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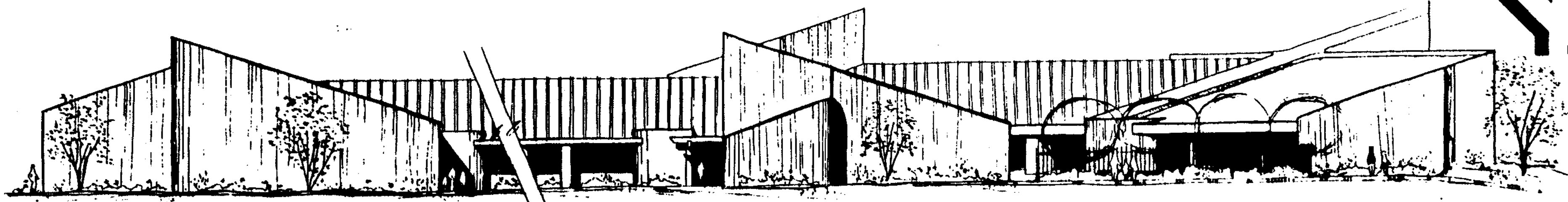
First
Presbyterian Church
GRAND JUNCTION, COLORADO

REVISED FINAL PLAN

PROJECT
9003

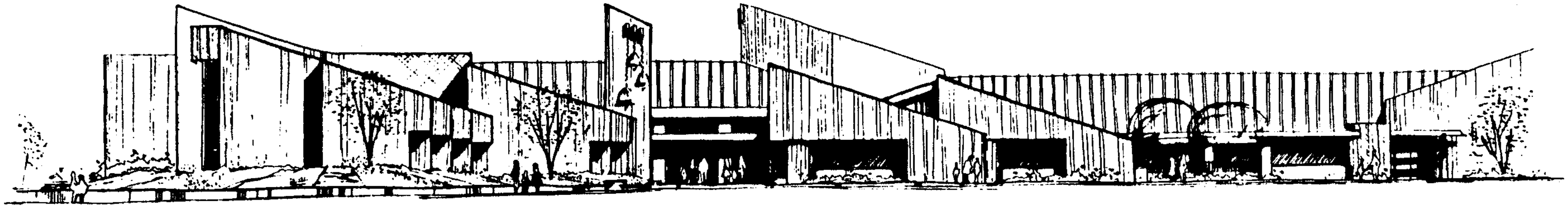
RECK
AND
ASSOCIATES
Architecture
Planning
Interior Design
9605 W. 49th Ave.
Wheat Ridge
Colorado
431-8600

SHEET 2 OF 4



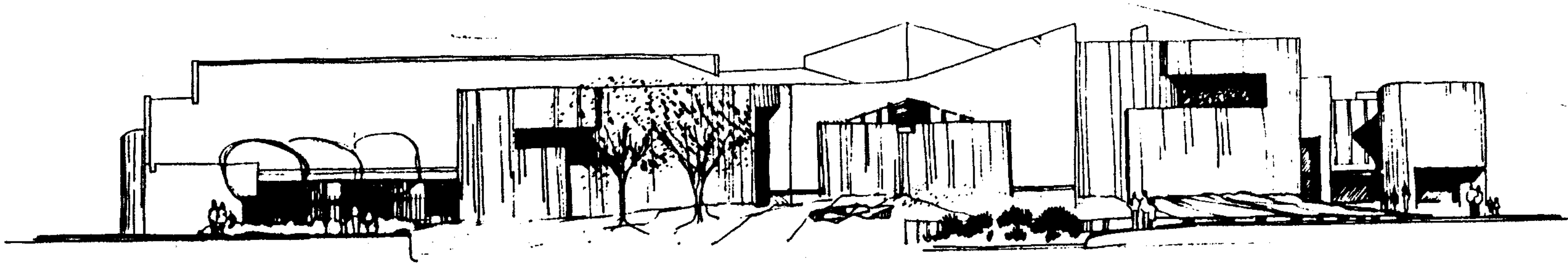
NORTH ELEVATION - FULL BUILD-OUT

3/32" : 1'-0"



SOUTH ELEVATION - FULL BUILD-OUT

3/32" : 1'-0"



WEST ELEVATION - FULL BUILD-OUT

3/32" : 1'-0"

1990

#43 90

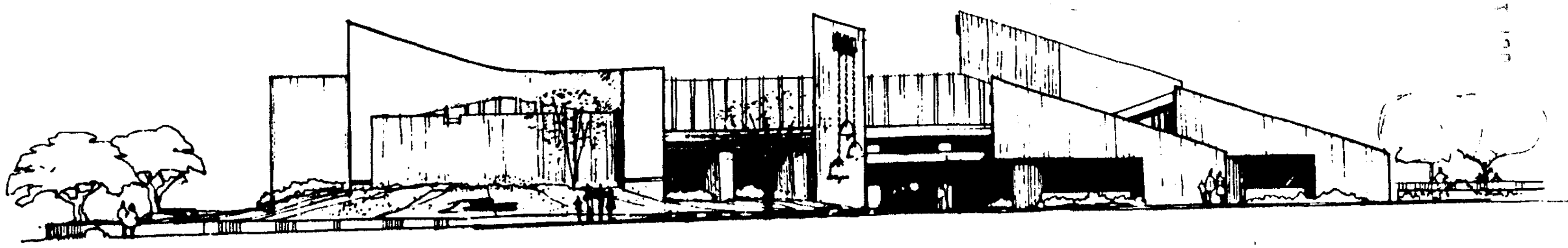
PROJECT
9003

REVISED FINAL PLAN

First Presbyterian Church
GRAND JUNCTION, COLORADO

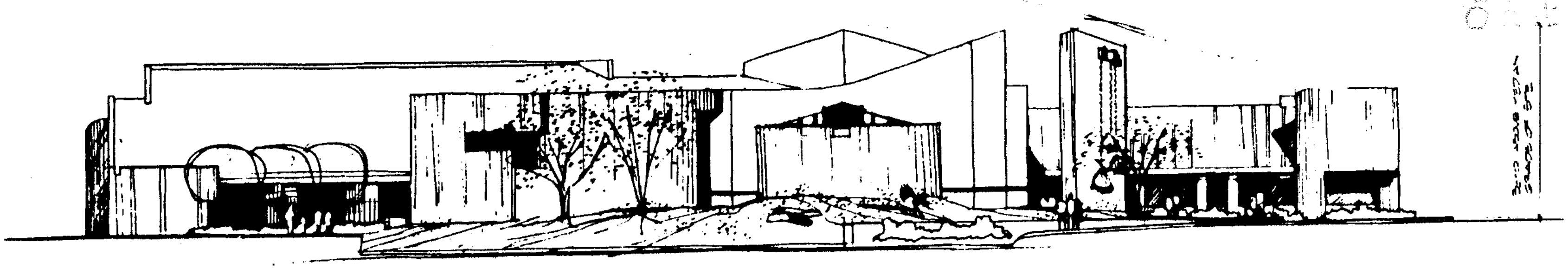
RECK
AND
ASSOCIATES
Architecture
Planning
Interior Design
9605 W. 49th Ave.
Wheat Ridge
Colorado
431-8600

SHEET
4
OF 4
DATE 10/1/90



SOUTH ELEVATION - PHASE ONE

3/32" : 1'-0"



WEST ELEVATION - PHASE ONE

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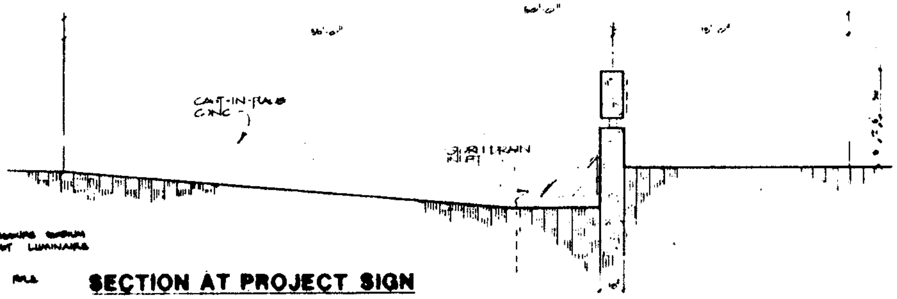
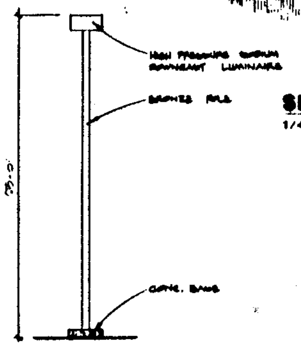
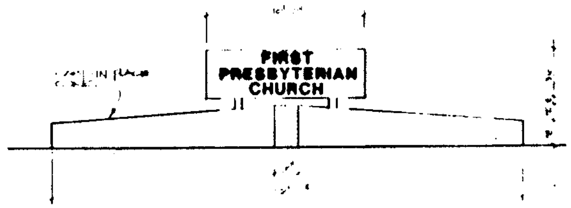


SECTION AT PROJECT SIGN

1/4" : 1'-0"



POLE LIGHT



ELEVATION AT PROJECT SIGN

1/4" : 1'-0"

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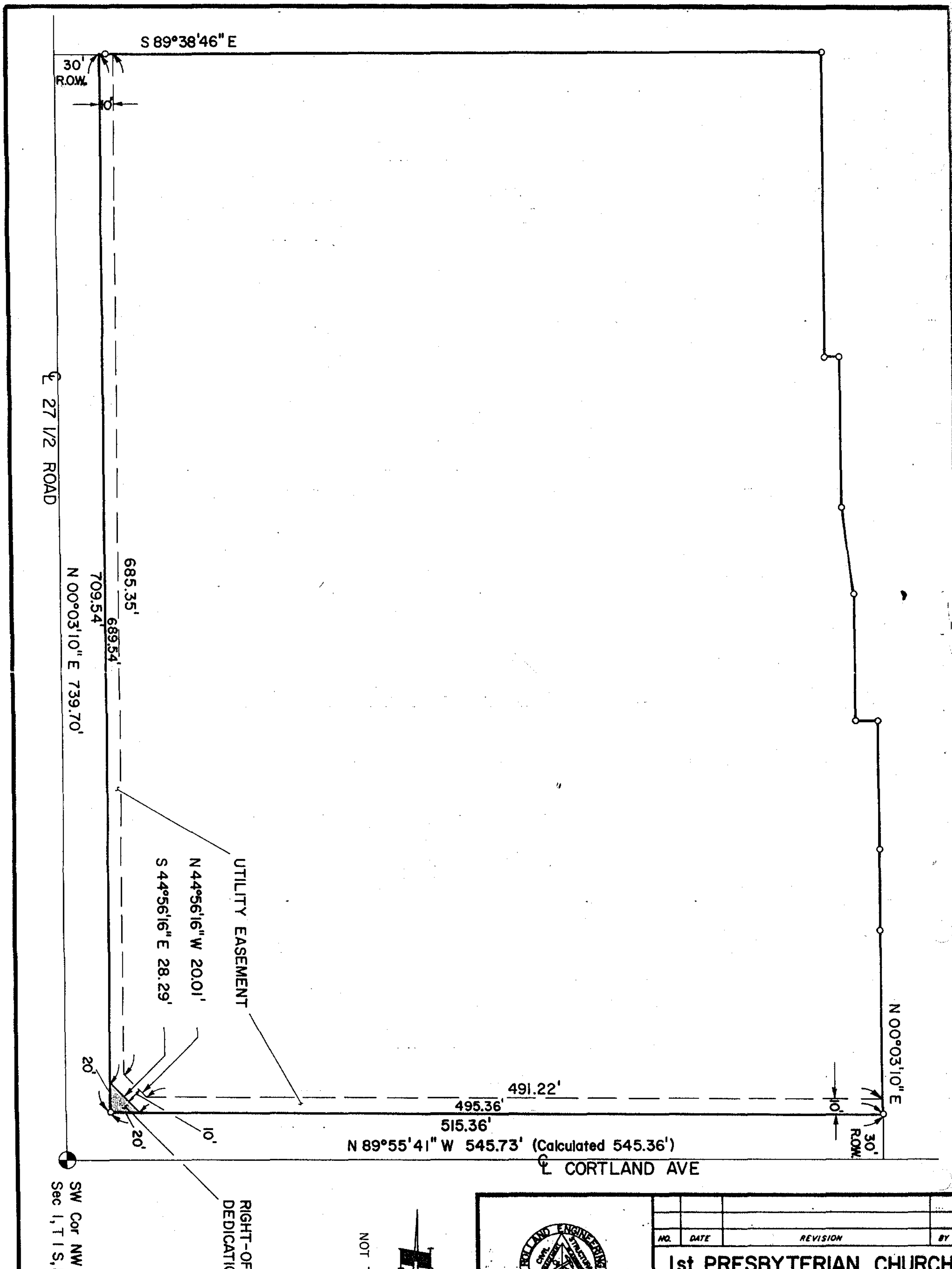
REVISED FINAL PLAN

First Presbyterian Church
GRAND JUNCTION, COLORADO

PROJECT
9003

RECK
AND
ASSOCIATES
Architecture
Planning
Interior Design
9605 W. 49th Ave.
Wheat Ridge
Colorado
431-6600

SHEET
3
OF 4
DATE 10/1/90



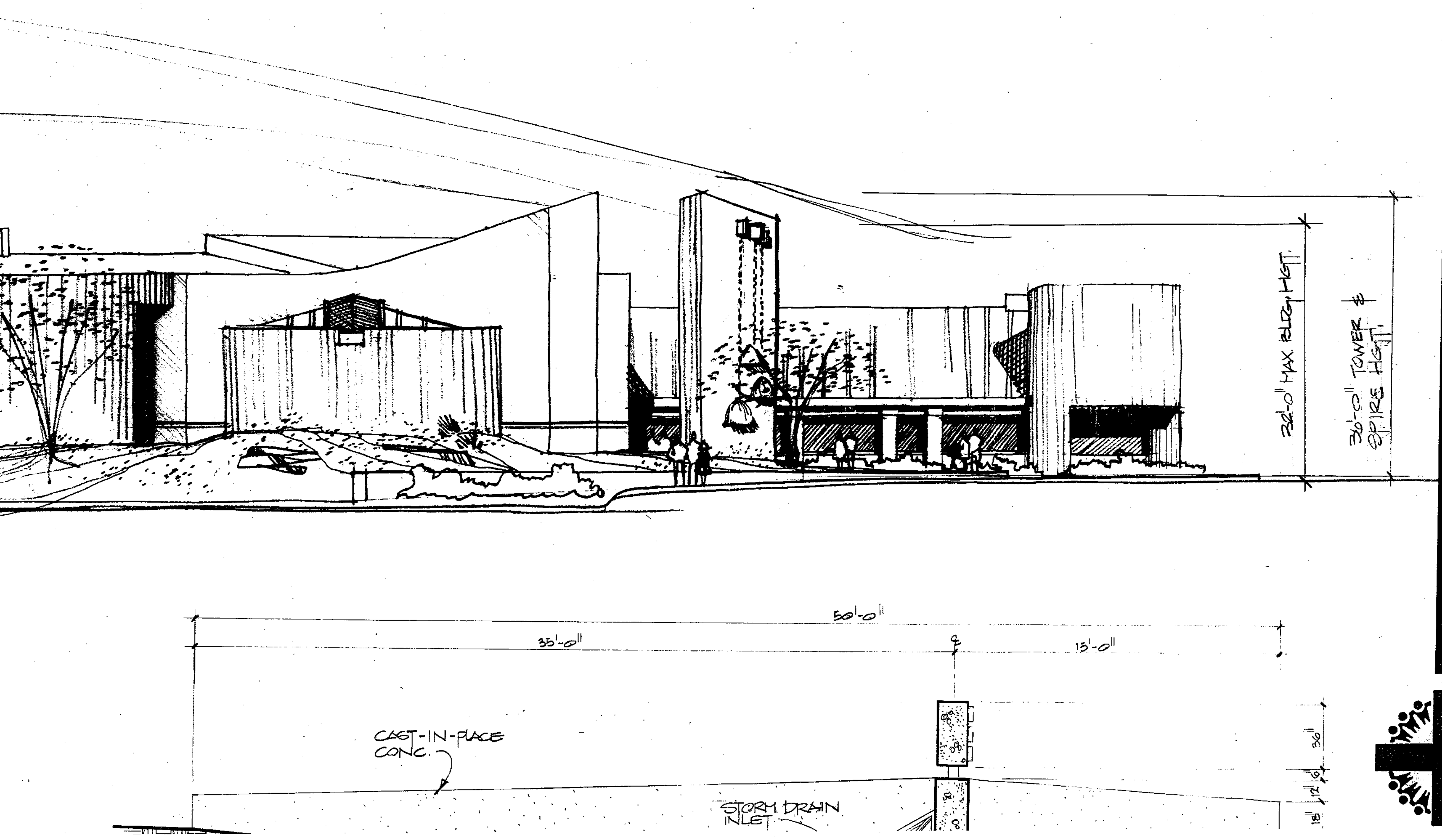
NO.	DATE	REVISION	BY

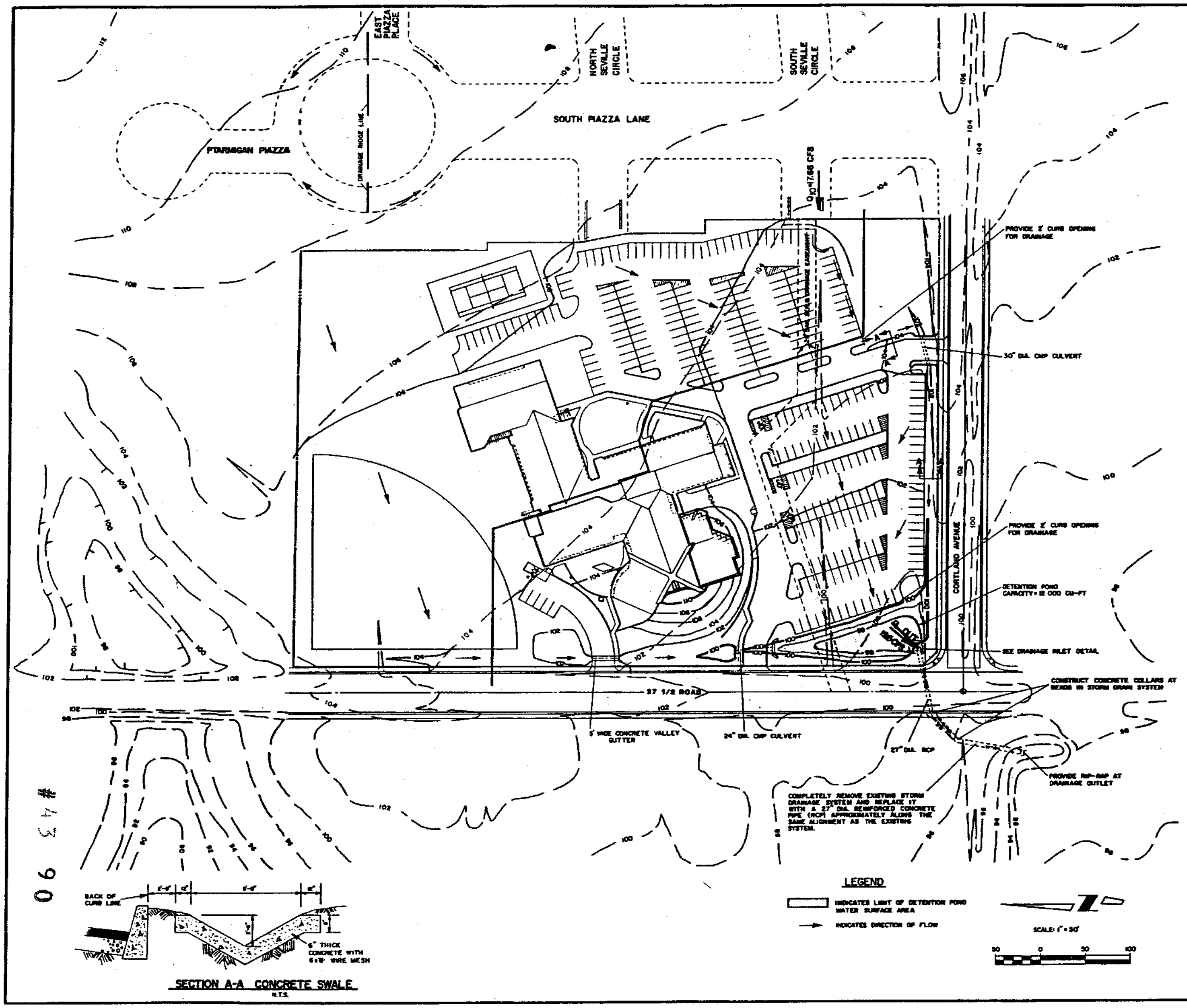
1st PRESBYTERIAN CHURCH
UTILITY EASEMENT
R.O.W. DEDICATION

GRAND JUNCTION, COLORADO

ROLLAND ENGINEERING
518 28 Road
Suite B-103
Grand Junction, Colorado 81501
(303) 243-8300

DESIGNED MDY	CHECKED TDR	JOB NO.	SHEET 1
DRAWN CFO	DATE 10/91		OF 1


32'-0" MAX. BLDG. HGT.
36'-0" TOWER & SPIRE HGT.
50'-0"
35'-0"
15'-0"
CAST-IN-PLACE CONC.
STORM DRAIN INLET
36"
6"
12"
18"



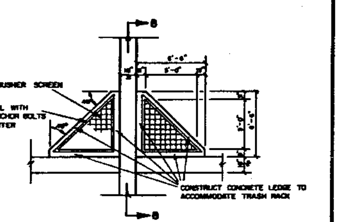
FIRST PRESBYTERIAN CHURCH

HISTORIC C = 0.30 · COMPOSITE C = 0.60
HISTORIC TC = 12.1 MIN. · DEVELOPED TC = 5.0 MIN.
ROOF AREA = 0.96 AC. · STORM FREQ = 10 YEARS
TOTAL AREA = 8.57 AC.

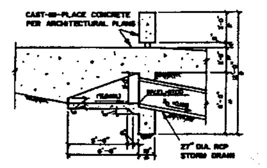
DURATION (MIN.)	INTENSITY (IN/HR)	PEAK Q (CFS)*	STORM VOL. (CU-FT)	RELEASE VOL. (CU-FT)	STORAGE (CU-FT)
5	3.30	35.42	10 626	5 850	4 776
10	2.60	31.65	18 990	11 700	7 290
15	2.15	29.23	26 308	17 550	8 758
20	1.85	27.62	33 144	23 400	9 744
25	1.65	26.54	39 810	29 250	10 560
30	1.50	25.73	46 314	35 100	11 214
35	1.35	24.93	52 353	40 950	11 403
40	1.20	24.12	57 888	46 800	11 088
45	1.10	23.58	63 666	52 650	11 016

NOTE: * INDICATES THAT THE DEVELOPED Q_{10} = 17.66 CFS CONTRIBUTION FROM APPLE CREST SUBDIVISION HAS BEEN ADDED TO THE PEAK Q_{10} VALUES OF THE FIRST PRESBYTERIAN CHURCH SITE.

MAX. STORAGE = 11 403 CU-FT
CRITICAL DURATION = 35 MIN.
RELEASE RATE = 19.5 CFS



DRAINAGE INLET DETAIL
N.T.S.



SECTION B-B DRAINAGE INLET
N.T.S.

& PER ARCHITECTURAL PLANS

NO.	DATE	REVISION	BY

FIRST PRESBYTERIAN CHURCH
DRAINAGE / GRADING PLAN

GRAND JUNCTION, COLORADO

ROLLAND ENGINEERING
518 28 Road
Suite B-105
Grand Junction, Colorado 81501
(303) 243-8300

DESIGNED: MGY · CHECKED: TDR · DRAWN: CFG · DATE: 9/90 · JOB NO. · SHEET 1

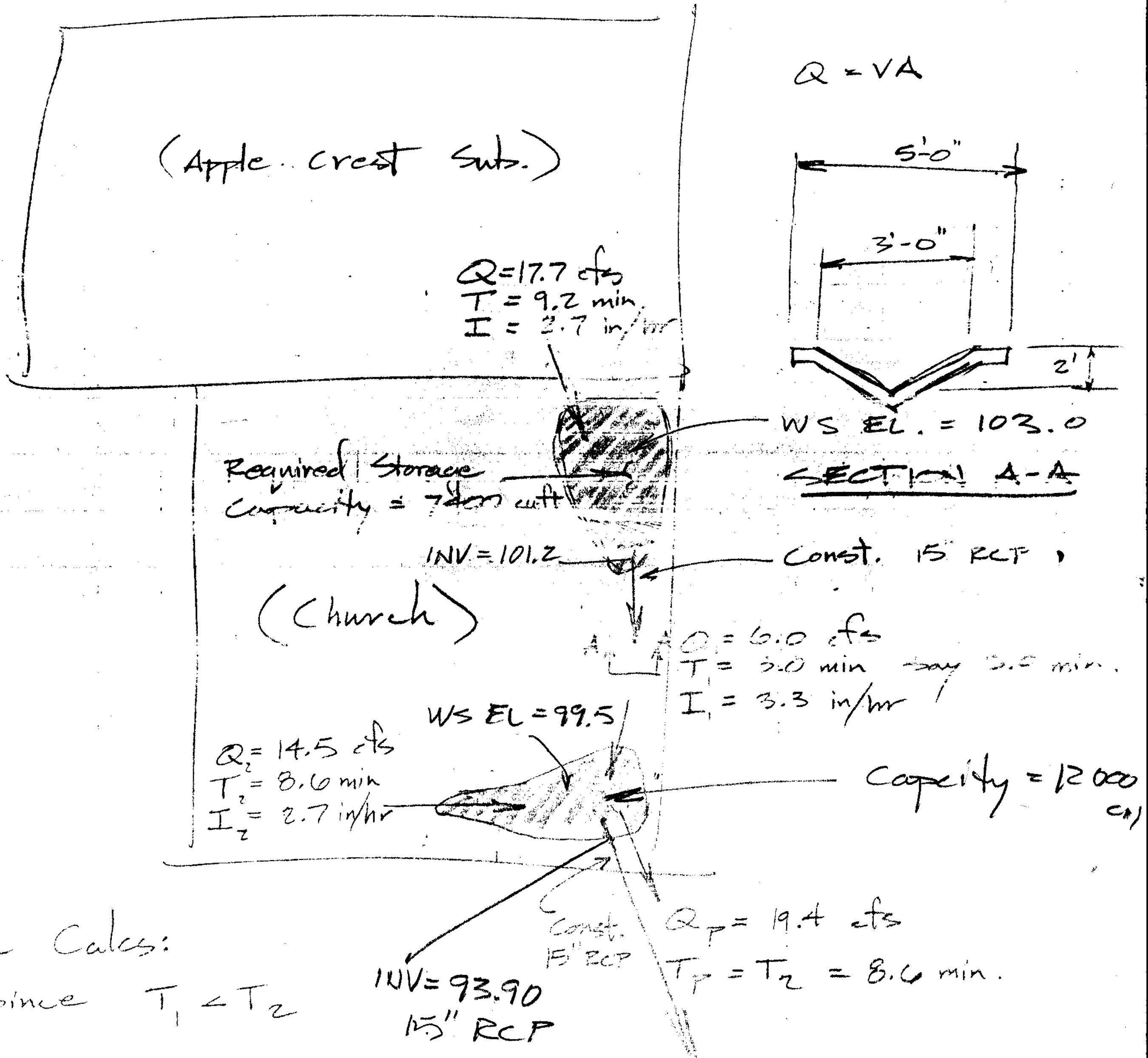
43 90

OCT 1 1990

Original
Do NOT Remove
From Office

PROPOSED SITE (S)
First Presbyterian Church

church
10/29/90



Confluence Calcs:

Since $T_1 < T_2$

$$\therefore Q_P = Q_2 + \left(\frac{I_2}{I_1}\right) Q_1 \quad 5.6$$

Q_{REL} = 13.5 cfs

$$= 14.5 + \left(\frac{2.7}{3.3}\right)(6.0)$$

$$= \underline{19.4 \text{ cfs}}$$

APPLE CREST SUBDIVISION

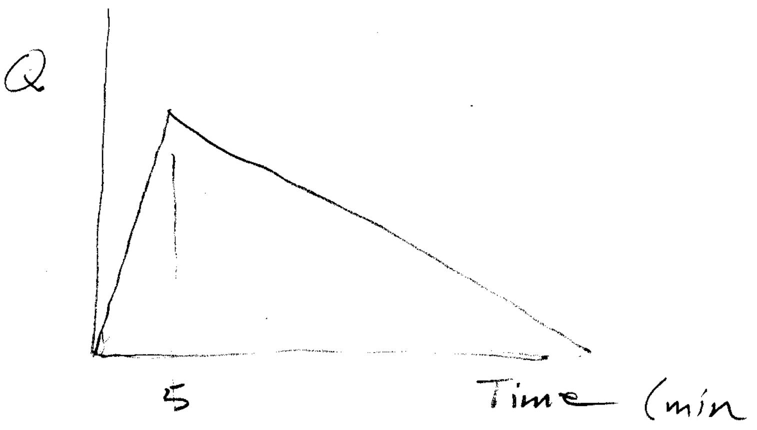
Composite C = 0.54

Area = 12.11

Release Rate = 6.0 cfs

Duration (Min.)	Intensity (In/Hr.)	Peak Q (cfs)	Storm Vol. (cu-ft)	Release Vol. (cu-ft)	Storage (cu-ft)
5	3.30	21.58	6 474	1 800	4 674
10	2.60	17.00	10 200	3 600	6 600
15	2.15	14.06	12 654	5 400	7 254
20	1.85	12.10	14 520	7 200	$\boxed{7\ 320}$
25	1.65	10.79	16 185	9 000	7 185
30	1.50	9.81	17 658	10 800	6 858
35	1.35	8.83	18 543	12 600	5 943
40	1.20	7.85	18 840	14 400	4 440
45	1.10	7.19	19 413	16 200	3 213
50	1.05	6.87	20 610	18 000	[illegible]
55	0.95	6.21	20 493	19 800	693
60	0.90	5.8[illegible]	21 204	21 600	-0-

$Q = CIA$



composite C = 0.60
Area (church) = 8.97

Release Rate = 13.5 cfs

Duration (Min.)	Intensity (In/Hr.)	Peak Q* (cfs)	Storm Vol. (cu-ft)	Release Vol. (cu-ft)	Storage (cu-ft)
5	3.30	23.76	7 128	4 050	3 078
10	2.60	19.99	11 994	8 100	$\boxed{3 894}$
15	2.15	17.57	15 813	12 150	3 663
20	1.85	15.96	19 152	16 200	2952
25	1.65	14.88	22 320	20 250	2 070
30	1.50	14.07	25 326	24 300	1 026
35	1.35	13.27	27 867	28 350	0
40	1.20	12.46	29 904	32 400	
45	1.10	11.92	32 184	36 450	
50	1.05	11.65	34 950		
55	0.95	11.11	36 663		
60	0.90	10.84	39 024		

* Add Q_{outlet} = 6.0 cfs to Q_{site} (church)

REF.: DESIGN GUIDELINES FOR STORM WATER MANAGEMENT IN MESA COUNTY, COLORADO

FIRST PRESBYTERIAN CHURCH: 9/25/90

HYDROLOGY - FOR UNDEVELOPED SITE

DESIGN FOR 10-YEAR STORM

$Q = CIA$

A) DETERMINE RUNOFF COEFFICIENT (C):

FROM TABLE 2-2 pg 2-3 OF STORM WATER MANAGEMENT
USE $C = 0.30$

B) DETERMINE TIME OF CONCENTRATION (T_c):

FROM FIG. 2-2 OF STORM WATER MANAGEMENT
USE $V_{ave} = 1.15$ ft/sec
WHERE OVERLAND FLOW LENGTH = 835'

$$\therefore T_c = \left(835/1.15\right)\left(\frac{1 \text{ min}}{60 \text{ sec}}\right) = 12.1 \text{ min.}$$

C) DETERMINE STORM INTENSITY (I):

FROM FIG. 2-1 OF STORM WATER MANAGEMENT
USE $I = 2.4$ in/hr FOR 10-YEAR STORM

D) DETERMINE SITE HISTORICAL RUNOFF (Q_H):

$Q_H = 0.30(2.4)(8.97) = 6.5$ cfs WHERE $A = 8.97$ ac

FIRST PRESBYTERIAN CHURCH: 9/25/90

HYDROLOGY – FOR DEVELOPED SITE

DESIGN FOR 10-YEAR STORM

$Q = CIA$

A) DETERMINE RUNOFF COEFFICIENT (C):

FROM TABLE 2-2: Lawns $C = 0.15$, Roofs $C = 0.90$, Asphalt/Conc. $C = 0.85$

COMPOSITE $C = \dfrac{[0.15(3.30) + 0.90(0.96) + 0.85(4.71)]}{8.97} = \underline{\underline{0.60}}$

B) DETERMINE TIME OF CONCENTRATION (T_c):

FROM FIG. 2-2

USE $V_{ave} = 2.0$ ft/sec

WHERE OVERLAND FLOW LENGTH = 1030'

$\therefore T_c = (1030/2.0)\left(\frac{1 \text{ min}}{60 \text{ sec}}\right) = \underline{8.6 \text{ min.}}$

C) DETERMINE STORM INTENSITY (I):

FROM FIG. 2-1

USE $I = \underline{2.7 \text{ in/hr}}$ FOR 10-YEAR STORM

D) DETERMINE SITE DEVELOPED RUNOFF (Q_D):

$Q_D = 0.60(2.7)(8.97) = 14.5$ cfs WHERE $A = \underline{8.97}$ Ac

APPLE CREST SUBDIVISION (CROWN HTS.) 9/25/90

HYDROLOGY - FOR DEVELOPED SITE

DESIGN FOR 10-YEAR STORM

$Q = CIA$

A) DETERMINE RUNOFF COEFFICIENT (C):

FROM TABLE 2-2: Lawns $C = 0.15$, Roofs $C = 0.90$

Asphalt/Conc. $C = 0.85$ common $C = 0.60$

$$\text{COMPOSITE } C = \frac{[0.15(5.08) + 0.90(1.79) + 0.85(4.10) + 0.60(1.14)]}{12.11}$$

$$= \underline{0.54}$$

B) DETERMINE TIME OF CONCENTRATION (T_c):

FROM FIG. 2-2

USE $V_{ave} = \underline{2.0 \text{ ft/sec}}$

WHERE OVERLAND FLOW LENGTH $= \underline{1100'}$

$$\therefore T_c = (1100/2.0)(1 \text{ min.}/60 \text{ sec}) = \underline{9.2 \text{ MIN.}}$$

C) DETERMINE STORM INTENSITY (I):

FROM FIG. 2-1

USE $I = \underline{2.7 \text{ in/hr}}$ FOR 10-YEAR STORM

D) DETERMINE SITE DEVELOPED RUNOFF (Q_D):

$$Q_D = 0.54(2.7)(12.11) = \underline{17.7 \text{ cfs}}$$

#43 90

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

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GEOTECHNICAL ENGINEERING STUDY

PROPOSED FIRST PRESBYTERIAN CHURCH

GRAND JUNCTION, COLORADO

Prepared for:

RECK AND ASSOCIATES

OCT 1 1990

PROJECT NUMBER: M90140GE

September 25, 1990

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M90140GE

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1.0 INTRODUCTION

This report presents the results of the geotechnical engineering study we conducted for the proposed First Presbyterian Church, Grand Junction, Colorado. The study was conducted at the request of Mr. Don Watkins, Reck and Associates.

The conclusions, suggestions and recommendations presented in this report are based on the data gathered during our site and laboratory study and on our experience with similar soil conditions. Factual data gathered during the field and laboratory work are summarized in Appendices A and B.

1.1 Proposed Construction

It is our understanding that the proposed structure will be single story structure and it may be supported on reinforced concrete foundations. Portions of the superstructure will be steel frame and a portion will be masonry walls. A basement may be included in the proposed structure. The floor may be concrete slab-on-grade floor or floor supported over a crawl space area.

1.2 Scope of Services

Our services included geotechnical engineering field and laboratory studies, and analysis and report preparation for the proposed site. The scope of our services is outlined below.

- The field study consisted of describing and sampling the soils encountered in six (6) auger advanced test borings in the proposed building area and three (3) auger advanced test borings in the parking lot area.

- The soils encountered in the test borings were described and samples retrieved for the subsequent laboratory study.

- The laboratory study included tests of select soil samples obtained during the field study to help assess the strength and swell/consolidation potential of the soils tested. A soil sample was tested for sulfate chemicals which may be potentially corrosive to concrete.

- This report presents our geotechnical engineering suggestions and recommendations for planning and design of site development including:

  - Viable foundation types for the conditions encountered,
  - Allowable bearing pressures for the foundation types,
  - Lateral earth pressure recommendations for design of laterally loaded walls,
  - Geotechnical considerations and recommendations for concrete slab-on-grade floors, and
  - Flexible Pavement thickness design recommendations.

- Our recommendations and suggestions are based on the subsoil and ground water conditions encountered during our site and laboratory studies.

2.Ø TECHNICAL GUIDE FOR DESIGN TEAM

This report contains geotechnical engineering suggestions and recommendations with background and support information. Design specific values may be difficult to locate quickly within the sections that present each design criteria. Therefore, some of the design values are discussed briefly in this section. The values presented here are a brief synopsis of the design values presented in the appropriate sections of this report and therefore do not present all of the pertinent information for that section.

The design soil bearing capacity will depend on the minimum depth of embedment of the bottom of the footing below the lowest adjacent grade and ranges from 1500 to 2000 pounds per square foot, with a minimum depth of embedment of one (1) to three (3) feet and a minimum dead load of 800 pounds per square feet. The soil bearing capacity may be increased by about 20 percent for transient loads such as wind and seismic loads. Foundation design considerations are presented in section 5.0.

Driven piles may be used to support the structure. The pile capacity is a function of pile type used, hammer used to install the piles, support characteristics of the material supporting the piles and the design load on the piles. We anticipate pile lengths of about twenty (20) to twenty five (25) feet when supported by the formational material underlying the site. Piles are discussed in section 6.3.

Drilled pier foundations should be drilled a minimum of five (5) feet into the hard unweathered formational material and designed for end bearing only using an end bearing capacity of 20,000 pounds per square foot and a minimum dead load of 5000 pounds per square foot. Drilled pier foundations are discussed in section 6.5.

Concrete slab-on-grade floors should be separated from all bearing members and placed on a blanket of compacted structural fill which is at least two (2) feet thick. We suggest the floor slab be reinforced with a 6 x 6 - W2.9 x W2.9 (6 x 6 - 6 x 6) welded wire mesh as a minimum reinforcement. Concrete floor slabs should be jointed with jointed areas about 200 square feet and approximately square. Concrete floor slabs are discussed in section 7.0.

Lateral earth pressures for the design of basement walls are; active lateral earth pressure of 65 pounds per cubic foot per foot of depth, at rest lateral earth pressure of 85 pounds per cubic foot per foot of depth, passive lateral earth pressure of 250 pounds per cubic foot per foot of embedment and a coefficient of friction between the concrete and soil of 0.25 for the natural on-site soils. Lateral earth pressures are discussed in section 9.0.

3.0 SITE CHARACTERISTICS

Site characteristics include observed existing and pre-existing site conditions that may influence the geotechnical engineering aspects of the proposed site development.

3.1 Site Location

The proposed building site is located in the southeast quadrant of the intersection of Cortland Avenue and 27 1/2 Road. A project vicinity map is presented on Figure 1.

3.2 Site Conditions

At the time of our field study the proposed building site was vacant except for an existing house and garage located in the northwest corner of the lot. The site slopes down to the northeast with about five (5) to six (6) feet of topographic relief across the proposed building site. An irrigation ditch is located along the south edge of the proposed building site.

3.3 Subsurface Conditions

The subsurface exploration consisted of observing, describing and sampling the soils encountered in six (6) test borings in the proposed building site and three (3) test borings in the proposed parking lot area. The approximate locations of the test borings are shown on Figure 2. The logs describing the soils encountered in the test borings are presented in Appendix A.

The soils encountered in the test borings consisted generally of clay with varying amounts of sand and shale fragments to a depth of about sixteen (16) to twenty two and one half (22 1/2) feet. The sandy clay soils tested have a moderate swell potential when wetted and may consolidate under light building loads.

Formational material was encountered in the test borings at a depth of about sixteen (16) to twenty two and one half (22 1/2) feet. The formational material encountered was a silty clay shale of the Mancos Formation. The Mancos shale typically has a low swell potential when in its hard unweathered condition, but may have a very high swell potential when only slightly weathered.

Free subsurface water was encountered in the test borings at a depth of about nine (9) to twelve (12) feet at the time of our field study.

4.Ø ON-SITE DEVELOPMENT CONSIDERATIONS

We anticipate the subsurface water elevation may fluctuate with seasonal and other varying conditions. Deep excavations may encounter subsurface water and soils that may tend to cave. It may be necessary to dewater construction excavations to provide more suitable working conditions. Excavations should be well braced or sloped to prevent wall collapse. Federal, state and local safety codes should be observed.

The formational material encountered in the test borings was very hard. We anticipate that it may be possible to excavate this material, however additional effort may be necessary. We do not recommend blasting to aid in excavation of the material. Blasting may fracture the formational material which will reduce the integrity of the support characteristics of the formational material.

It has been our experience that sites in developed areas may contain existing subterranean structures or poor quality man-placed fill. If subterranean structures or poor quality man-placed fill are suspected or encountered, they should be removed and replaced with compacted structural fill as discussed under COMPACTED STRUCTURAL FILL below.

5.0 FOUNDATION DISCUSSION

Two criteria for any foundation which must be satisfied for satisfactory foundation performance are:

1) contact stresses must be low enough to preclude shear failure of the foundation soils which would result in lateral movement of the soils from beneath the foundation, and

2) settlement or heave of the foundation must be within amounts tolerable to the superstructure.

The soils encountered in the test borings have varying engineering characteristics that may influence the design and construction considerations of the foundations. The characteristics include swell potential, settlement potential, bearing capacity and the bearing conditions of the soils supporting the foundations. These are discussed below.

5.1 Swell Potential

Some of the materials encountered in the test borings at the anticipated foundation depth may have swell potential. Swell potential is the tendency of the soil to increase in volume when it becomes wetted. The volume change occurs as moisture is absorbed into the soil and water molecules become attached to or adsorbed by the individual clay platlets. Associated with the process of volume change is swell pressure. The swell pressure is the force the soil applies on its surroundings when moisture is absorbed into the soil. Foundation design considerations concerning swelling soils include structure tolerance to movement and dead load pressures to help restrict uplift. The structure's tolerance to movement should be addressed by the structural engineer and is dependent upon many facets of the design including the overall structural concept and the building material. The uplift forces or pressure due to wetted clay soils can be addressed by designing the foundations with a minimum dead load. Suggestions and recommendations for design dead load are presented below.

5.2 Settlement Potential

Settlement potential of a soil is the tendency for a soil to experience volume change when subjected to a load. Settlement is characterized by downward movement of all or a portion of the supported structure as the soil particles move closer together resulting in decreased soil volume. Settlement potential is a function of foundation loads, depth of footing embedment, the width of the footing and the settlement potential or compressibility of the influenced soil. Foundation design considerations concerning settlement potential include the amount of movement tolerable to the structure and the design and construction concepts to help reduce the potential movement. The settlement potential of the foundation can be reduced by reducing foundation pressures and/or placing the foundations on a blanket of compacted structural fill. The anticipated post construction settlement potential and suggested compacted fill thickness recommendations are based on site specific soil conditions and are presented below.

5.3 Soil Support Characteristics

The soil bearing capacity is a function of the engineering properties of the soils supporting the foundations, the foundation width, the depth of embedment of the bottom of the foundation below the lowest adjacent grade, the influence of the ground water and the amount of settlement tolerable to the structure. Soil bearing capacity and associated minimum depth of embedment are presented below.

6.0 FOUNDATION RECOMMENDATIONS

We have analyzed spread footings, drilled piers and driven piles as potential foundation systems for the proposed structure. These are discussed below.

6.1 Spread Footings

The structure may be founded on spread footings which are placed either on the natural undisturbed soils or a blanket of compacted structural fill. The blanket of compacted structural fill is to help reduce the anticipated post construction settlement. The anticipated post construction settlement and associated fill thickness supporting the footings are presented below. If the footings are supported on a blanket of compacted structural fill the blanket of compacted structural fill should extend beyond each edge of each footing a distance at least equal to the fill thickness. Geotechnical engineering recommendations for constructing compacted structural fill are presented below. The soil bearing capacity will depend on the minimum depth of embedment of the bottom of the footing below the lowest adjacent grade. The embedment concept is shown on Figure 3. The soil bearing capacity and associated minimum depth of embedment are tabulated below.

SOIL BEARING CAPACITY (POUNDS PER SQUARE FOOT)	MINIMUM DEPTH OF EMBEDMENT OF THE BOTTOM OF THE FOOTING BELOW THE LOWEST ADJACENT GRADE (FEET)
1500	1
1750	2
2000	3

The soil bearing capacity may be increased by about 20 percent for transient loads such as wind and seismic loads.

The anticipated post construction settlement may be reduced by placing the footings on a blanket of compacted structural fill. The anticipated post construction settlement and associated thickness of compacted structural fill are presented below.

THICKNESS OF COMPACTED STRUCTURAL FILL SUPPORTING FOOTINGS	ANTICIPATED CONSTRUCTION SETTLEMENT (INCHES)
Ø	about 3/4
B/2	about 1/2
B	about 1/4

*B is equal to the footing width

6.2 General Spread Footing Considerations

In our analysis it was necessary to assume that the material encountered in the test borings extended throughout the building site and to a depth below the maximum depth of the influence of the footings. We should be contacted to observe the soils exposed in the foundation excavations prior to placement of foundations to verify the assumptions made during our analysis.

We anticipate that the surface of the formational material may undulate which may result in a portion of the footings supported on the overlying soils. If this happens the foundations will perform differently between the areas supported on formational material and the areas supported on the non-formational material. For this reason we suggest that if formational material is encountered only in portions of the foundation excavations at footing depth the foundation in all areas should be extended to support the structure on the formational material.

The bottom of any footings exposed to freezing temperatures should be placed below the maximum depth of frost penetration for the area. Refer to the local building code for details.

The bottom of the foundation excavations should be proof rolled or proof compacted prior to placing compacted structural fill or foundation concrete. The proof rolling is to help reduce the influence of any disturbance that may occur during the excavation operations. Any areas of loose, low density or yielding soils evidenced during the proof rolling operation should be removed and replaced with compacted structural fill. Caution should be exercised during the proof rolling operations. Excess proof rolling may increase pore pressure of the soil and degrade the integrity of the soils.

All footings should be proportioned as much as practicable to reduce the post construction differential settlement. Footings for large localized loads should be designed for bearing pressures and footing dimensions in the range of adjacent footings to reduce the potential for differential settlement. We are available to discuss this with you.

Foundation walls should be reinforced, for geotechnical purposes, with at least two (2) number 5 bars, continuous at the top and the bottom (4 bars total), at maximum vertical spacing. This will help provide the walls with additional beam strength and help reduce the effects of slight differential settlement. The walls may need additional reinforcing steel for structural purposes. The structural engineer should be consulted for foundation design.

6.3 Piles

Piles should be designed as end bearing piles in the formational material. Pile capacity is a function of the pile type chosen, equipment used to install the piles, installation procedure and building loads on the piles. The pile types that are suitable for this project are discussed below.

Steel H-piles have proved successful for pile installations where the piles extend to a hard bearing stratum such as the formational shales encountered in our test borings. The steel H-piles will withstand hard driving with limited damage and are easily handled. "H" piles may be spliced without loss of bending strength and point reinforcement may be used to prevent tip damage when driving through boulders or obstacles. Prefabricated splices and point reinforcement are available.

For design purposes and budget estimates we suggest you consider steel H-piles about ten (10) inches across, such as 10 x 42 or 10 x 57, extending about one (1) to three (3) feet into the hard unweathered formational material which will result in piles to about twenty (20) to twenty five (25) feet below the existing ground surface. We anticipate that the surface of the formational material may undulate. The pile length may be variable. H-Piles can be typically designed for loads of about forty (40) to sixty (60) tons each. For pile groups to support concentrated loads we suggest spacing individual piles no closer than three (3) diameters to each other spaced on centers.

Pipe piles will carry heavy loads when founded on a high bearing capacity stratum, such as the formational material underlying the site. Prefabrication splices and point reinforcement are available for pipe piles.

For design and budgeting purposes we suggest that you consider pipe piles about eight (8) to ten (1Ø) inches in diameter driven closed end, and backfilled with concrete. The concrete backfill will allow reinforcing steel to be cast into the pile to tie the pile and structure together easily. We anticipate that pipe piles will be about twenty (2Ø) to twenty five (25) feet long below the existing ground surface and typically can be designed to support fifty (5Ø) to one hundred (1ØØ) tons per pile. Pile clusters or groups for concentrated loads should be spaced no closer than three (3) diameters to each other, center to center.

6.4 Piles-General Considerations

The structural engineer should be consulted for structural requirements of the piles. Once a pile type, hammer and contractor has been selected we should be contacted for specific geotechnical design and construction criteria. We suggest that the piles be installed with a pile driving hammer that has a minimum rated energy of 2Ø,ØØØ foot pounds per stroke. Any tendency for the piles to deviate from their required driving alignment during the installation operations should be corrected at the on set of the deviation.

We suggest that the pile set used to determine the bearing depth of the pile be several blows per inch greater than the set determined by an appropriate dynamic formula. This is to help reduce the potential for post construction settlement of the piles. We are available to assess the set/load criteria once the pile type and specific hammer are chosen.

Pile splices made during the driving operation may result in delays of the driving and may allow sufficient time for the pore pressures incurred during driving to dissipate and cause difficulties in completion of the driving of the pile. We suggest that splices made during the driving operation be kept to a minimum. If needed, splices should be made prior to driving to provide appropriate length piles.

We suggest that your geotechnical consultant be present during the installation of the piles to provide geotechnical engineering consultation and provide a pile driving record for each pile installed for the as-built records. We are available to discuss this with you.

6.5 Drilled Piers

Drilled piers or caissons that are drilled into the unweathered formational material may be used to support the proposed structure. The piers should be drilled into the unweathered formational material a distance equal to at least two

(2) pier diameters, or ten (1Ø) feet, whichever is deeper. The piers should be designed as end bearing piers using a formational material bearing capacity of 2Ø,ØØØ pounds per square foot and a side friction of 2,ØØØ pounds per square foot for the portion of the pier in the unweathered formational material. We suggest that piers be designed using end bearing capacity only. The side shear may be used for the design to resist uplift forces. When using skin friction for resisting uplift we suggest that you discount the upper portion of the pier embedment in the formational material to a depth of at least one and one half (1 1/2) pier diameters into the formational material.

The bottom of the pier holes should be cleaned to insure that all loose and disturbed materials are removed prior to placing pier concrete. Because of the rebounding potential in the formational materials when unloaded by excavation and the possibility of desiccation of the newly exposed material we suggest that concrete be placed in the pier holes immediately after excavation and cleaning.

If the piers are designed and constructed as discussed above we anticipate that the post construction settlement potential of each pier may be less than about one quarter (1/4) inch.

The portion of the pier above the formational surface and in the weathered formational material should be cased with a sono tube or similar casing to help prevent flaring on the top of the pier holes and help provide a positive separation of the pier concrete and the adjacent soils. Construction of the piers should include extreme care to prevent flaring of the top of the piers. This is to help reduce the potential of the soils should swelling occur, to "grab" the top of the pier causing uplift forces which will put the pier in tension. The drilled piers should be vertically reinforced to provide tensile strength in the piers should swelling on-site soils apply tensile forces on the piers. The structural engineer should be consulted to provide structural design recommendations.

The grade beams between caissons should be provided with void spaces between the soil and the grade beam. The grade beam should not come in contact with the soils. This is to help reduce the potential for heave of the foundations should the soils swell.

Free ground water and caving soils were encountered in the test borings at the time of the field study. We anticipate that ground water will be encountered in the pier holes. If ground water is encountered, the pier holes should be dewatered prior to placing pier concrete and no pier concrete should be placed when more than six (6) inches of water exists in the bottom of the pier holes. The piers should be filled with a tremie placed concrete immediately after the drilling and cleaning operation is complete. It may be necessary to case the pier holes with temporary casing to prevent caving during pier construction.

Very difficult drilling conditions were encountered in the formational material during our field study. We anticipate that the formational material may be very difficult to drill with pier drilling readily available in western Colorado. It may be necessary to obtain specialty pier drilling equipment to drill piers into the formational material encountered in our test borings.

The structural engineer should be consulted to provide structural design recommendations for the drilled piers and grade beam foundation system.

7.0 INTERIOR FLOOR SLAB DISCUSSION

It is our understanding that, as currently planned, the floor may be either a concrete slab-on-grade or a supported structural floor. The natural soils that will support interior floor slabs are stable at their natural moisture content. However, the owner should realize that when wetted, the site soils may experience volume changes.

Engineering design dealing with swelling soils is an art which is still in its infancy. The owner is cautioned that the soils on this site may have swelling potential and concrete slab-on-grade floors and other lightly loaded members may experience movement when the supporting soils become wetted. We suggest you consider floors suspended from the foundation systems as structural floors or a similar design that will not be influenced by subgrade volume changes. If the owner is willing to accept the risk of possible damage from swelling soils supporting concrete slab-on-grade floors, the following recommendations to help reduce the damage from swelling soils should be followed. These recommendations are based on generally accepted design and construction procedures for construction on soils that tend to experience volume changes when wetted and are intended to help reduce the damage caused by swelling soils. Lambert and Associates does not intend that the owner, or the owner's consultants should interpret these recommendations as a solution to the problems of swelling soils, but as measures to reduce the influence of swelling soils.

Concrete flatwork, such as concrete slab-on-grade floors, should be underlain by compacted structural fill. The layer of compacted fill should be at least two (2) feet thick and constructed as discussed under COMPACTED STRUCTURAL FILL below.

The natural soils exposed in the areas supporting concrete slab-on-grade floors should be kept very moist during construction prior to placement of concrete slab-on-grade floors. This is to help increase the moisture regime of the potentially expansive soils supporting floor slabs and help reduce the expansion potential of the soils. We are available to discuss this concept with you.

Concrete slab-on-grade floors should be provided with a positive separation, such as a slip joint, from all bearing members and utility lines to allow their independent movements and to help reduce possible damage that could be caused by movement of soils supporting interior slabs. The floor slab should be constructed as a floating slab. All water and sewer pipe lines should be isolated from the slab. Any appliances, such as a water heater or furnace, placed on the floating floor slab should be constructed with flexible joints to accommodate future movement of the floor slab with respect to the structure. We suggest partitions constructed on the concrete slab-on-grade floors be provided with a void space above or below the partitions to relieve stresses induced by elevation changes in the floor slab.

The concrete slabs should be scored or jointed to help define the locations of any cracking. The areas defined by scoring and jointing should be about square and enclose about 2ØØ square feet. Also, joints should be scored in the floors a distance of about three (3) feet from, and parallel to, the walls.

If moisture rise through the concrete slab-on-grade floors will adversely influence the performance of the floor or floor coverings a moisture barrier may be installed beneath the floor slab to help discourage capillary and vapor moisture rise through the floor slab. The moisture barrier may consist of a heavy plastic membrane, six (6) mil or greater, protected on the top and bottom by at least two (2) inches of clean sand. The plastic membrane should be lapped and taped or glued and protected from punctures during construction.

The Portland Cement Association suggests that welded wire reinforcing mesh is not necessary in concrete slab-on-grade floors when properly jointed. It is our opinion that welded wire mesh may help improve the integrity of the slab-on-grade floors. We suggest that concrete slab-on-grade floors should be reinforced, for geotechnical purposes, with at least 6 x 6 - W2.9 x W2.9 (6 x 6 - 6 x 6) welded wire mesh positioned midway in the slab. The structural engineer should be contacted for structural design of the floor slabs.

8.Ø COMPACTED STRUCTURAL FILL

Compacted structural fill is typically a material which is constructed for direct support of structures or structural components.

There are several material characteristics which should be examined before choosing a material for potential use as compacted structural fill. These characteristics include; the size of the larger particles, the engineering characteristics of the fine grained portion of material matrix, the moisture content that the material will need to be for compaction with respect to the existing initial moisture content, the organic content of the material, and the items that influence the cost to use the material.

Compacted fill should be a non-expansive material with the maximum aggregate size less than about two (2) to three (3) inches and less than about twenty five (25) percent coarser than three quarter (3/4) inch size. The reason for this maximum size is that larger sizes may have too great an influence on the compaction characteristics of the material and may also impose point loads on the footings or floor slabs that are in contact with the material. Frequently pit-run material or crushed aggregate material is used for structural fill material. Pit-run material may be satisfactory, however crushed aggregate material with angular grains is preferable. Angular particles tend to interlock with each other better than rounded particles.

The fine grained portion of the fill material will have a significant influence on the performance of the fill. Material which has a fine grained matrix composed of silt and/or clay which exhibits expansive characteristics should be avoided for use as structural fill. The moisture content of the material should be monitored during construction and maintained near optimum moisture content for compaction of the material.

Soil with an appreciable organic content may not perform adequately for use as structural fill material due to the compressibility of the material and ultimately due to the decay of the organic portion of the material.

The natural on-site soils are not suitable for use as compacted structural fill material supporting building or structure members because of their clay content and swell potential. The natural on-site soils may be used as compacted fill in areas that will not influence the structure such as to establish general site grade. We are available to discuss this with you.

All areas to receive compacted structural fill should be properly prepared prior to fill placement. The preparation should include removal of all organic or deleterious material and the areas to receive fill should be proof rolled after the organic deleterious material has been removed. Any areas of soft, yielding, or low density soil, evidenced during the proof rolling operation should be removed. Fill should be moisture conditioned, placed in thin lifts and compacted to at least 90 percent of maximum dry density as defined by ASTM D1557, modified Proctor.

We recommend that the geotechnical engineer or his representative be present during the proof rolling and fill placement operations to observe and test the material.

9.0 LATERAL EARTH PRESSURES

Laterally loaded walls supporting soil, such as basement walls, will act as retaining walls and should be designed as such.

Walls that are designed to deflect and mobilize the internal soil strength should be designed for active earth pressures.

Walls that are restrained so that they are not able to deflect to mobilize internal soil strength should be designed for at-rest earth pressures. The values for the lateral earth pressures will depend on the type of soil retained by the wall, backfill configuration and construction technique. We suggest that for design of laterally loaded walls you consider an active lateral earth pressure of 65 pounds per cubic foot per foot of depth and an at-rest lateral earth pressure of 85 pounds per cubic foot per foot of depth for the on-site soils retained.

The soils tested have measured swell pressure of about 1000 pounds per square foot. Our experience has shown that the actual swell pressure may be much higher. If the retained soils should be come moistened after construction the soil may swell against retaining or basement walls. The walls should be designed to resist the swell pressure of the soils.

The above lateral earth pressures may be reduced by overexcavating the wall backfill area beyond the zone of influence and backfilling with crushed rock type material. The zone of influence concept is presented on Figure 4. We suggest that you consider, if the backfill areas are overexcavated beyond the zone of influence and backfilled with crushed rock type material, an active lateral earth pressure of 35 pounds per cubic foot of depth and an at-rest lateral earth pressure of 50 pounds per cubic foot per foot of depth for the design of laterally loaded walls.

Resistant forces used in the design of the walls will depend on the type of soil that tends to resist movement. We suggest that you consider a passive earth pressure of 25Ø pounds per cubic foot per foot of embedment and a coefficient of friction of Ø.25 for the on-site soil.

The lateral earth pressure values provided above, for design purposes, should be treated as equivalent fluid pressures. The lateral earth pressures provided above are for level well drained backfill and do not include surcharge loads or additional loading as a result of compaction of the backfill. Unlevel or non-horizontal backfill either in front of or behind walls retaining soils will significantly influence the lateral earth pressure values. Care should be taken during construction to prevent construction and backfill techniques from overstressing the walls retaining soils. Backfill should be placed in thin lifts and compacted, as discussed in this report to realize the lateral earth pressure values.

Walls retaining soil should be designed and constructed so that hydrostatic pressure will not accumulate or will not affect the integrity of the walls. Drainage plans should include a subdrain behind the wall at the bottom of the backfill to provide positive drainage. Exterior retaining walls should be provided with weep holes to help provide an outlet for collected water behind the wall. The ground surface adjacent to the wall should be sloped to permit rapid drainage of rain, snow melt and irrigation water away from the wall backfill. Sprinkler systems should not be installed directly adjacent to retaining or basement walls.

10.0 DRAIN SYSTEM

Free ground water was encountered in the test borings at a depth of about nine (9) to twelve (12) feet. We anticipate that the ground water elevation may be higher during wetted seasons. A drain system should be provided around building spaces below the finished grade and behind any walls retaining soil. The drain systems are to help reduce the potential for hydrostatic pressure to develop behind retaining walls. A sketch of the drain system is shown on Figure 5.

Subdrains should consist of a three (3) or four (4) inch diameter perforated pipe surrounded by a filter. The filter should consist of a filter fabric or a graded material such as washed concrete sand or pea gravel. If sand or gravel is chosen the pipe should be placed in the middle of about four (4) cubic feet of aggregate per linear foot of pipe. The drain system should be sloped to positive gravity outlets. If the drains are daylighted the drains should be provided with all water outlets and the outlets should be maintained to prevent them from being plugged or frozen. If the drains are graded to sumps or pump discharge we suggest that the pump be sized to pump the maximum anticipated flow and the sump be equipped with a backup pump for added protection in the event of primary pump failure. The sump should be equipped with a high water alarm to alert of a pump failure. We should be called to observe the soil exposed in the excavations and to verify the details of the drain system.

11.Ø FLEXIBLE PAVEMENT THICKNESS DESIGN RECOMMENDATIONS

The alternate pavement sections tabulated below are based assumed traffic information and the subgrade resistance value (R-Value) obtained from test results of samples retrieved from the site. The R-Value was calculated from a California Bearing Ratio (CBR) of 3 using "Thickness Design-Asphalt Pavements for Highways and Streets", by The Asphalt Institute, Manual Series Number 1, (MS-1) dated September, 1981. An R-Value of 5 was used in our analysis.

PARKING AREA PAVEMENT DESIGN SECTIONS
BASED ON NO TRUCK OR BUS TRAFFIC

ASPHALT CONCRETE (INCHES)	CLASS 6 OR EQUIVALENT AGGREGATE BASE COURSE (INCHES)	CLASS 2 OR EQUIVALENT AGGREGATE BASE COURSE (INCHES)	RECONDITIONED SUBGRADE (INCHES)
2	8 1/2	Ø	12
2	4	6	12
3	5	O	12
4 1/2	Ø	Ø	12

TRAVEL LANE PAVEMENT DESIGN SECTIONS
BASED ON SOME TRUCK OR BUS TRAFFIC

ASPHALT CONCRETE (INCHES)	CLASS 6 OR EQUIVALENT AGGREGATE BASE COURSE (INCHES)	CLASS 2 OR EQUIVALENT AGGREGATE BASE COURSE (INCHES)	RECONDITIONED SUBGRADE (INCHES)
2	12	Ø	12
2	5	9	12
3	8 1/2	Ø	12
3	5	4 1/2	12
5 1/2	Ø	Ø	12

The pavement design sections of less than three (3) inches of asphalt over aggregate base course presented may be used, although, because of the shorter life before maintenance and the relatively poorer long term performance, we suggest that this be considered as an intermediate design section only. If this design section is used we suggest you consider a later asphalt overlay of about one (1) to one and one half (1 1/2) inches to extend the life of the pavement section. The overlay should be constructed prior to any visible distress occurring in the pavement.

We suggest that the construction of the pavement section be done after the completion of other construction activities on the site. The reason for this is that the above sections are not designed to accommodate high frequency heavy vehicle loads which are often associated with construction operations.

Prior to the construction of the pavement section the areas for pavement should be stripped of vegetation, existing fill, debris or any deleterious materials. The natural, subgrade soils exposed by stripping operations, should be scarified to a depth of at least six (6) inches and replaced with compacted fill to subgrade elevation or scarified to one (1) foot below subgrade elevation and recompacted, whichever will provide at least one (1) foot of reconditioned subgrade soil. The subgrade soil should be moisture conditioned prior to compaction and should be compacted to at least ninety (9Ø) percent of maximum dry density as defined by ASTM D1557, modified Proctor density.

The aggregate base course material and aggregate subbase course material should conform to Colorado State Highway Specifications for Class 6 and Class 2 or similar materials respectively. We recommend material testing of these products prior to their use to determine conformance with the specifications. The base course and subbase course materials should be moisture conditioned prior to compaction and individual lift thicknesses during compaction should not exceed six (6) inches. The base course and subbase course materials should be compacted to at least ninety (9Ø) percent of maximum dry density as defined by ASTM D1557, modified Proctor density.

Asphalt pavement materials should be mixed from an approved mix design stating the Marshall properties, optimum asphalt content, job mix formula, recommended mixing and placing temperatures, and the date of the mix design. We recommend verification of the mix design prior to paving. The asphalt materials should be placed in lifts not exceeding three (3) inches and compacted to a minimum of ninety five (95) percent of the Marshall density. Rolling patterns for compaction should be established during pavement construction to allow proper compaction.

12.Ø RIGID PAVEMENT SECTION DESIGN RECOMMENDATIONS

Analysis of a rigid concrete pavement section for the site was beyond our requested scope of services. We are available to perform this analysis at your request. Please contact us if you desire recommendations for rigid pavement design sections.

13.Ø BACKFILL

Backfill areas and utility trench backfill should be constructed such that the backfill will not settle after completion of construction, and that the backfill is relatively impervious for the upper few feet. The backfill material should be free of trash and other deleterious material. It should be moisture conditioned and compacted to at least 9Ø percent relative compaction using a modified Proctor density (ASTM D1557). Only enough water should be added to the backfill material to allow proper compaction. Do not pond, puddle, float or jet backfill soils.

14.Ø SURFACE DRAINAGE

The foundation soils should be prevented from becoming wetted after construction. This can be aided by providing positive and rapid drainage of surface water away from the building.

The final grade of the ground surface adjacent to the building should have a definite slope away from the foundation walls on all sides. We suggest a minimum fall of one about (1) foot in the first ten (1Ø) feet away from the foundation. Downspouts and faucets should discharge onto splash blocks that extend beyond the limits of the backfill areas. Splash blocks should be sloped away from the foundation walls. Snow storage areas should not be located next to the structure. Proper surface drainage should be maintained from the onset of construction through the proposed project life.

15.Ø LANDSCAPE IRRIGATION

An irrigation system should not be installed next to foundation walls, concrete flatwork or asphalt paved areas. If an irrigation system is installed, the system should be placed so that the irrigation water does not fall or flow near foundation walls, flatwork or pavements. The amount of irrigation water should be controlled.

16.Ø SOIL CORROSIVITY TO CONCRETE

Chemical tests were performed on a sample of soil obtained during the field study. The soil sample was tested for pH, water soluble sulfates, and total dissolved salts. The results are presented in Appendix B. The test results indicate a water soluble sulfate content of Ø.Ø15 percent. Based on the American Concrete Institute (ACI) information a water soluble sulfate content of Ø.Ø15 percent indicates mild exposure to sulfate attack on concrete. The American Concrete Institute does not indicate a type cement and a maximum water/cement ratio for concrete where mild exposure to sulfate attack will occur.

17.Ø CONCRETE QUALITY

It is our understanding current plans include reinforced structural concrete for building foundations and walls, and may include concrete slabs-on-grade and pavement. To insure concrete members perform as intended the structural engineer should be consulted and should address factors such as design loadings, anticipated movement and deformations.

The quality of concrete is influenced by proportioning of the concrete mix, placement, consolidation and curing. Desirable qualities of concrete include compressive strengths, water tightness and resistance to weathering. Engineering observations and testing of concrete during construction is essential as an aid to safeguard the quality of the completed concrete. Testing of the concrete is normally performed to determine compressive strength, entrained air content, slump and temperature. We recommend that your budget include provisions for testing of concrete during construction and that the testing consultant be retained by the owner or the owner's engineer or architect, not the contractor, to maintain third party credibility.

18.0 POST DESIGN CONSIDERATIONS

This subsoil and foundation study is based on limited sampling, therefore it is necessary to assume that the subsurface conditions do not vary greatly from those encountered in the test borings. Our experience has shown that significant variations are likely to exist and can become apparent only during additional on-site excavation. For this reason, and because of our familiarity with the project, Lambert and Associates should be retained to observe foundation excavations prior to foundation construction, to observe the geotechnical aspects of the construction, and to be available in the event any unusual or unexpected conditions are encountered. The cost of the geotechnical observations and testing during construction or additional engineering consultation is not included in the fee for this report. We recommend that your construction budget include site visits early during construction for the project geotechnical engineer to observe foundation excavations and for additional site visits to test compacted soil. We recommend that the observation and material testing services during construction be retained by the owner or the owner's engineer or architect, not the contractor, to maintain third party credibility. We have included a copy of a report prepared by Van Gilder Insurance which discusses testing services during construction. It is our opinion that the owner, architect and engineer be familiar with the information. If you have any questions regarding this concept please contact us.

It is difficult to predict if unexpected subsurface conditions will be encountered during construction. Since such conditions may be found we suggest that the owner and the contractor make provisions in their budget and construction schedule to accommodate unexpected subsurface conditions.

This report does not provide earthwork specifications. We can provide guidelines for our use in preparing project specific earthwork specifications. Please contact us if you need these for your project.

19.Ø LIMITATIONS

It is the owner's and the owner's representatives responsibility to read this report and become familiar with the recommendations and suggestions presented. We should be contacted if any questions arise concerning the geotechnical engineering aspects of this project as a result of the information presented in this report.

The recommendations outlined above are based on our understanding of the currently proposed construction. We are available to discuss the details of our recommendations with you, and revise them where necessary. This geotechnical engineering report is based on the proposed site development and scope of services as provided to us by Mr. Don Watkins, Reck and Associates, on the type of construction planned, existing site conditions at the time of the field study, and on our findings. Should the planned, proposed use of the site be altered, Lambert and Associates must be contacted, since any such changes may make our suggestions and recommendations given inappropriate. This report should be used ONLY for the planned development for which this report was tailored and prepared, and ONLY to meet information needs of the owner and the owner's representatives. In the event that any changes in the future, design or location of the building are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report are modified or verified in writing. It is recommended that the geotechnical engineer be provided the opportunity for a general review of the final project design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications.

This report presents both suggestions and recommendations. The suggestions are presented so that the owner and the owner's representatives may compare the cost to the potential risk or benefit for the suggested procedures.

We represent that our services were performed within the limits prescribed by you and with the usual thoroughness and competence of the current accepted practice of the geotechnical engineering profession in the area. No warranty or representation either expressed or implied is included or intended in this report or our contract. We are available to discuss our findings with you. If you have any questions please contact us. The supporting data for this report is included in the accompanying figures and appendices.

This report is a product of Lambert and Associates. Excerpts from this report used in other documents may not convey the intent or proper concepts when taken out of context or they may be misinterpreted or used incorrectly. Reproduction, in part or whole, of this document without prior written consent of Lambert and Associates is prohibited.

We have enclosed a copy of a brief discussion about geotechnical reports published by Association of Soil and Foundation Engineers for your reference.

Please call when further consultation or observations and tests are required.

If you have any questions concerning this report or if we may be of further assistance, please contact us.

Respectfully submitted;

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THE PROFESSIONAL LIABILITY PERSPECTIVE

Vol. 8, No. 8 | Copyright 1988 | August 1988

WHO HIRES THE TESTING LABORATORY?

It is one of those relatively small details in the overall scheme of things. Independent testing may be required by local building codes, or it may be insisted upon by lenders. Additional testing can usually be ordered by the design team during construction. Whatever the source of the requirement, many owners perceive it to be an unnecessary burden—an additional cost imposed principally for someone else's benefit.

What does this have to do with you? You may be the only one in a position to influence the use of testing and inspection services so they become more, rather than less likely to contribute to a successful outcome. There seems to be an almost irresistible inclination on the part of some owners to cast aside their potential value to the project in favor of the administrative and financial convenience of placing responsibility for their delivery into the hands of the general contractor.

Resist this inclination where you can. It is not in your client's best interests, and it is certainly not in yours. There are important issues of quality and even more important issues of life safety at stake. In the complex environment of today's construction arena, it makes very little sense for either of you to give up your control of quality control. Yet it happens altogether too often.

What's Behind this Misadventure?

The culprit seems to be the Federal Government. In the 1960's, someone came up with the idea that millions could be saved by eliminating the jobs of Federal workers engaged in construction inspection. The procurement model used to support this stroke of genius was the manufacturing segment of the economy, where producers of goods purchased by the Government had been required for years to conduct their own quality assurance programs. The result was a trendy new concept in Federal construction known as Contractor Quality Control (CQC).

It was a dumb idea. Costs were simply shifted from the Federal payroll to capital improvement budgets. Government contractors, selected on the basis of the lowest bid, were handed resources to assure the quality of their own performance. Some did so; many did not. All found themselves caught up in an impossible conflict between the demands of time and cost, on one hand, and the dictates of quality, on the other.

CQC was opposed by the Associated General Contractors of America, by independent testing laboratories, by the design professions, and by those charged with front-line responsibility for quality control in the Federal Agencies. Eventually, even the General Accounting Office came to the conclusion that it ought to be abandoned. But, once set in motion and fueled by the pervasive influence of the Federal Government, the idea spread—first to state and local governments; finally, to the private sector.

Why would the private sector embrace such an ill-conceived notion? Because so many owners view testing and inspection as an undertaking which simply duplicates something they are entitled to in any event. They are confident they will be protected by contract documents which cover every detail and contingency. They look to local building inspectors to assure compliance with codes. And they fully expect the design team to fulfill its obligation to safeguard the quality of the work.

A Fox in the Henhouse

If testing is perceived as little more than an 'unnecessary, but unavoidable expense, why not make the general contractor responsible for controlling the cost? It may produce a savings, and it certainly eliminates an adminstrative headache. If contractual obligations dealing with the project schedule and budget can be enforced, surely those governing quality can be enforced, as well. Possibly so, but who is going to do it?

Some testing consultants will not accept CQC work. The reasons they give come from firsthand experience. They include: 1) inadequate to barely adequate scope, 2) selection based on the lowest bid; 3) non-negotiable contract terms inappropriate to the delivery of a professional service; 4) intimidation of inspectors by field supervisors; and 5) suppression of low or failing test results. This ought to be fair warning to any owner.

Keeping Both Hands on the Wheel

The largest part of the problem, from your point of view, is one of artful persuasion. If you cannot convince your client of the value of independent testing and inspection, no one can. Yet, if you do not, you are likely to find yourself responsible for an assurance of quality you are in no position to deliver. How can you keep quality control where it belongs and, in the process, prevent the owner from compromising his or her interests in the project as well as yours? Consider these suggestions:

1. Put the issue on an early agenda. It needs your attention. Anticipate the owner's inclination to avoid dealing with testing and inspection, and explain its importance to the success of the project. Persist, if you can, until your client agrees to hire the testing laboratory independently and to establish an adequate budget to meet the anticipated costs. A testing consultant hired by the owner cannot be fired by the general contractor for producing less than favorable results.

2. Tailor the testing requirements carefully. Scissors and paste can be your very worst enemies. Specify what the job requires, retain control of selection and hiring, make certain the contractor's responsibilities for notification for scheduling purposes are clear, and require that copies of all reports be distributed by the laboratory directly to you.

3. Insist on a preconstruction testing conference. It can be an essential element of effective coordination. Include the owner, the general contractor, major subcontractors, the testing consultant, and the design team. Review your requirements, the procedures to be followed, and the responsibilities of each of the parties. Have the testing consultant prepare a conference memorandum for distribution to all participants.

4. Monitor tests and inspections closely. Make certain your field representative is present during tests and inspections, so that deficiencies in procedures or results can be reported and acted upon quickly. Scale back testing if it becomes clear it is appropiate to do so under the circumstances; do not hesitate to order additional tests if they are required.

5. Finally, keep your client informed. Without your help, he or she is not likely to understand what the test results mean, nor will your actions in response to them make much sense. If additional testing is called for, explain why. Remember, it is an unexpected and, possibly, unbudgeted additional cost for which you will need to pave the way. In this sense, independent testing and inspection can serve an important, secondary purpose. You might view it as a communications resource. Use it in this way, and it just may yield unexpected dividends.

IMPORTANT INFORMATION
ABOUT YOUR
GEOTECHNICAL ENGINEERING REPORT

More construction problems are caused by site subsurface conditions than any other factor. As troublesome as subsurface problems can be, their frequency and extent have been lessened considerably in recent years, due in large measure to programs and publications of ASFE/ The Association of Engineering Firms Practicing in the Geosciences.

The following suggestions and observations are offered to help you reduce the geotechnical-related delays, cost-overruns and other costly headaches that can occur during a construction project.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

A geotechnical engineering report is based on a subsurface exploration plan designed to incorporate a unique set of project-specific factors. These typically include: the general nature of the structure involved, its size and configuration; the location of the structure on the site and its orientation; physical concomitants such as access roads, parking lots, and underground utilities, and the level of additional risk which the client assumed by virtue of limitations imposed upon the exploratory program. To help avoid costly problems, consult the geotechnical engineer to determine how any factors which change subsequent to the date of the report may affect its recommendations.

Unless your consulting geotechnical engineer indicates otherwise, *your geotechnical engineering report should not be used:*

- When the nature of the proposed structure is changed, for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one;
- when the size or configuration of the proposed structure is altered;
- when the location or orientation of the proposed structure is modified;
- when there is a change of ownership, or
- for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems which may develop if they are not consulted after factors considered in their report's development have changed.

MOST GEOTECHNICAL "FINDINGS" ARE PROFESSIONAL ESTIMATES

Site exploration identifies actual subsurface conditions only at those points where samples are taken, when they are taken. Data derived through sampling and subsequent laboratory testing are extrapolated by geotechnical engineers who then render an opinion about overall subsurface conditions, their likely reaction to proposed construction activity, and appropriate foundation design. Even under optimal circumstances actual conditions may differ from those inferred to exist, because no geotechnical engineer, no matter how qualified, and no subsurface exploration program, no matter how comprehensive, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than a report indicates. Actual conditions in areas not sampled may differ from predictions. *Nothing can be done to prevent the unanticipated, but steps can be taken to help minimize their impact.* For this reason, *most experienced owners retain their geotechnical consultants through the construction stage,* to identify variances, conduct additional tests which may be needed, and to recommend solutions to problems encountered on site.

SUBSURFACE CONDITIONS CAN CHANGE

Subsurface conditions may be modified by constantly-changing natural forces. Because a geotechnical engineering report is based on conditions which existed at the time of subsurface exploration, *construction decisions should not be based on a geotechnical engineering report whose adequacy may have been affected by time.* Speak with the geotechnical consultant to learn if additional tests are advisable before construction starts.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

Geotechnical engineers' reports are prepared to meet the specific needs of specific individuals. A report prepared for a consulting civil engineer may not be adequate for a construction contractor, or even some other consulting civil engineer. Unless indicated otherwise, this report was prepared expressly for the client involved and expressly for purposes indicated by the client. Use by any other persons for any purpose, or by the client for a different purpose, may result in problems. *No individual other than the client should apply this report for its intended purpose without first conferring with the geotechnical engineer. No person should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.*

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical engineering report. To help avoid these problems, the geotechnical engineer should be retained to work with other appropriate design professionals to explain relevant geotechnical findings and to review the adequacy of their plans and specifications relative to geotechnical issues.

BORING LOGS SHOULD NOT BE SEPARATED FROM THE ENGINEERING REPORT

Final boring logs are developed by geotechnical engineers based upon their interpretation of field logs (assembled by site personnel) and laboratory evaluation of field samples. Only final boring logs customarily are included in geotechnical engineering reports. *These logs should not under any circumstances be redrawn* for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs, delays, disputes and unanticipated costs are the all-too-frequent result.

To minimize the likelihood of boring log misinterpretation, *give contractors ready access to the complete geotechnical engineering report* prepared or authorized for their use. Those who do not provide such access may proceed under the *mistaken* impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes which aggravate them to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY

Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical consultants. To help prevent this problem, geotechnical engineers have developed model clauses for use in written transmittals. These are *not* exculpatory clauses designed to foist geotechnical engineers' liabilities onto someone else. Rather, they are definitive clauses which identify where geotechnical engineers' responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report, and you are encouraged to read them closely. Your geotechnical engineer will be pleased to give full and frank answers to your questions.

OTHER STEPS YOU CAN TAKE TO REDUCE RISK

Your consulting geotechnical engineer will be pleased to discuss other techniques which can be employed to mitigate risk. In addition, ASFE has developed a variety of materials which may be beneficial. Contact ASFE for a complimentary copy of its publications directory.

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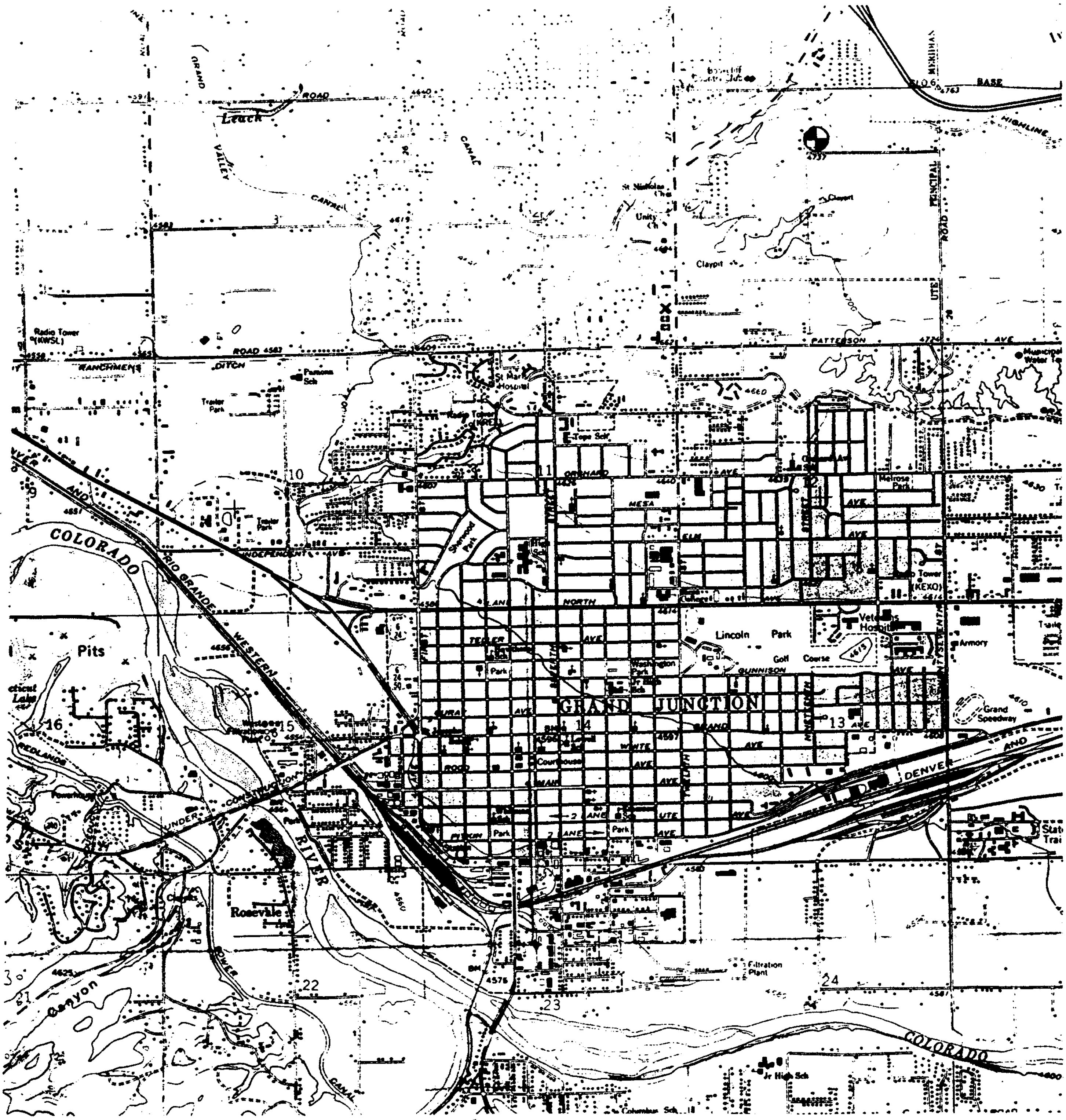


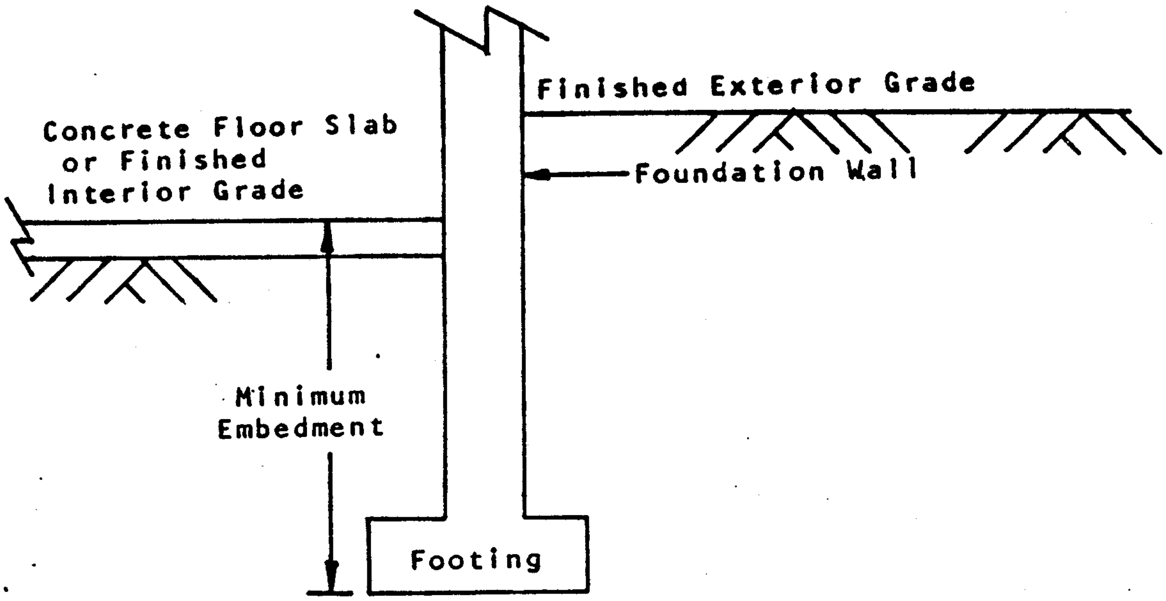
Indicates approximate project location

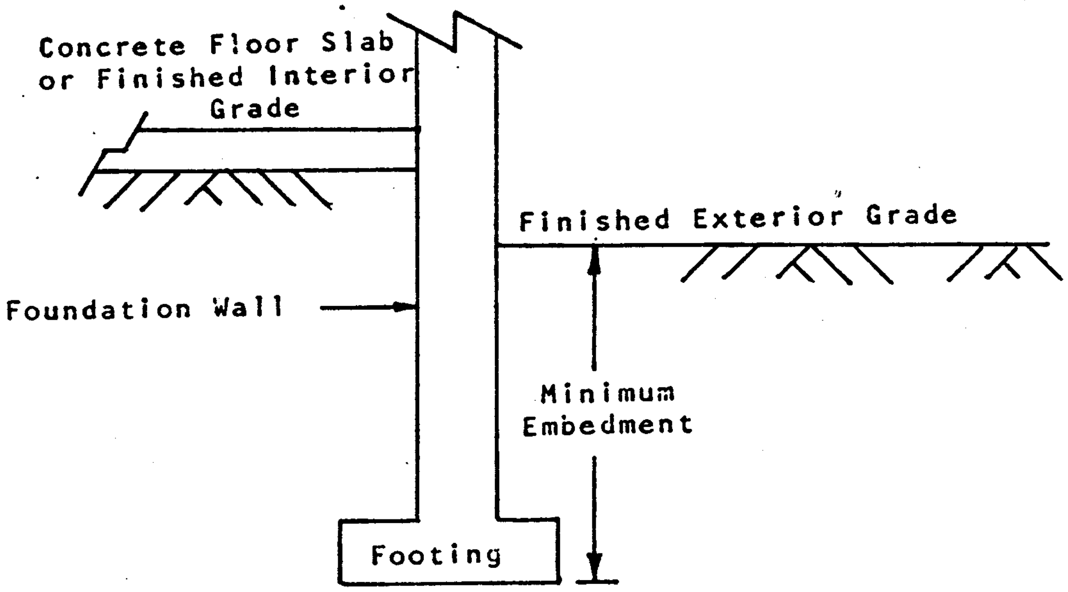
This map was reproduced from a map provided by United States Geologic Survey and is intended to present geotechnical data only

NO SCALE

PROJECT VICINITY MAP

Lambert and Associates	Project No.: M90140GE
	Date: 9/25/90
	Figure: 1





EMBEDMENT CONCEPT

NO SCALE

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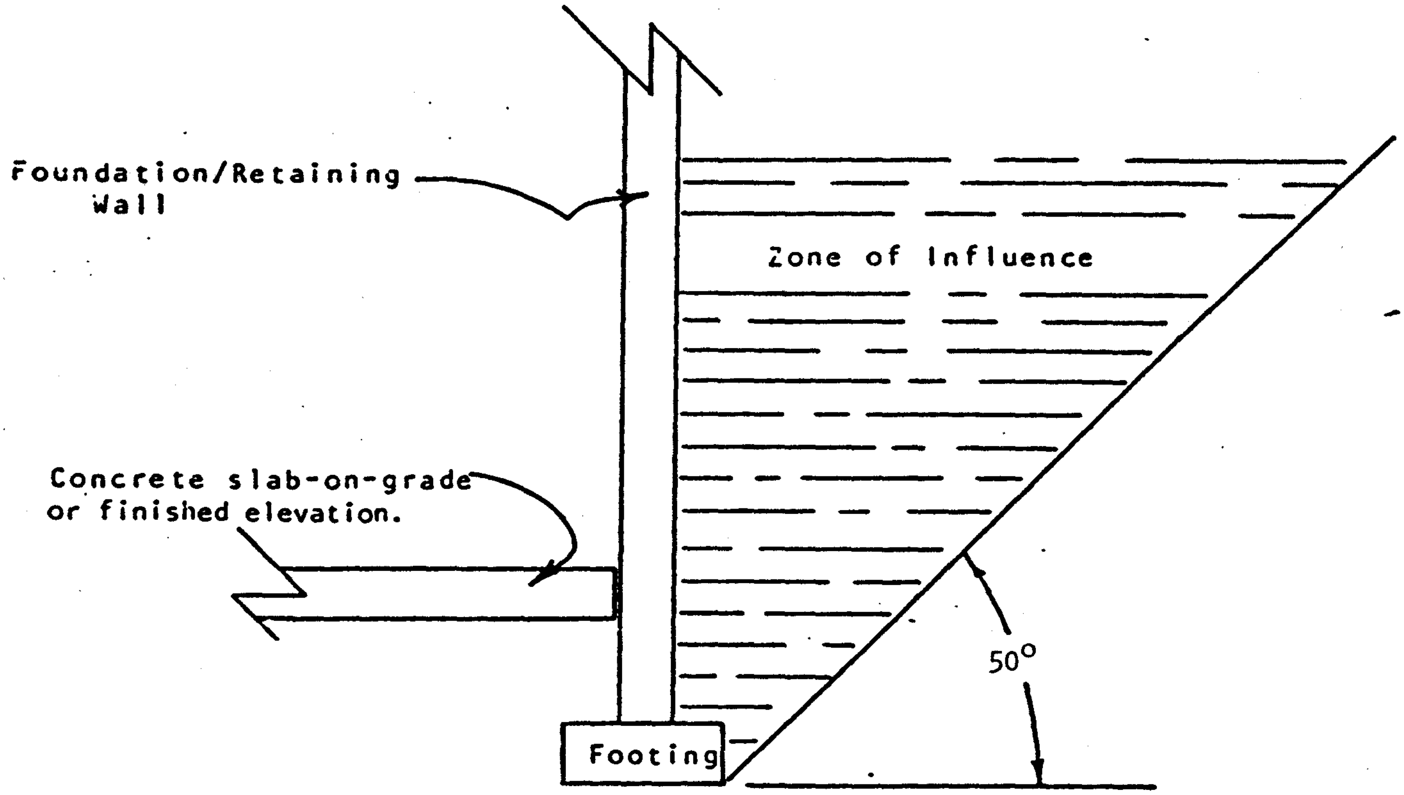
Figure:	3



SACKFILL ZONE OF INFLUENCE CONCEPT

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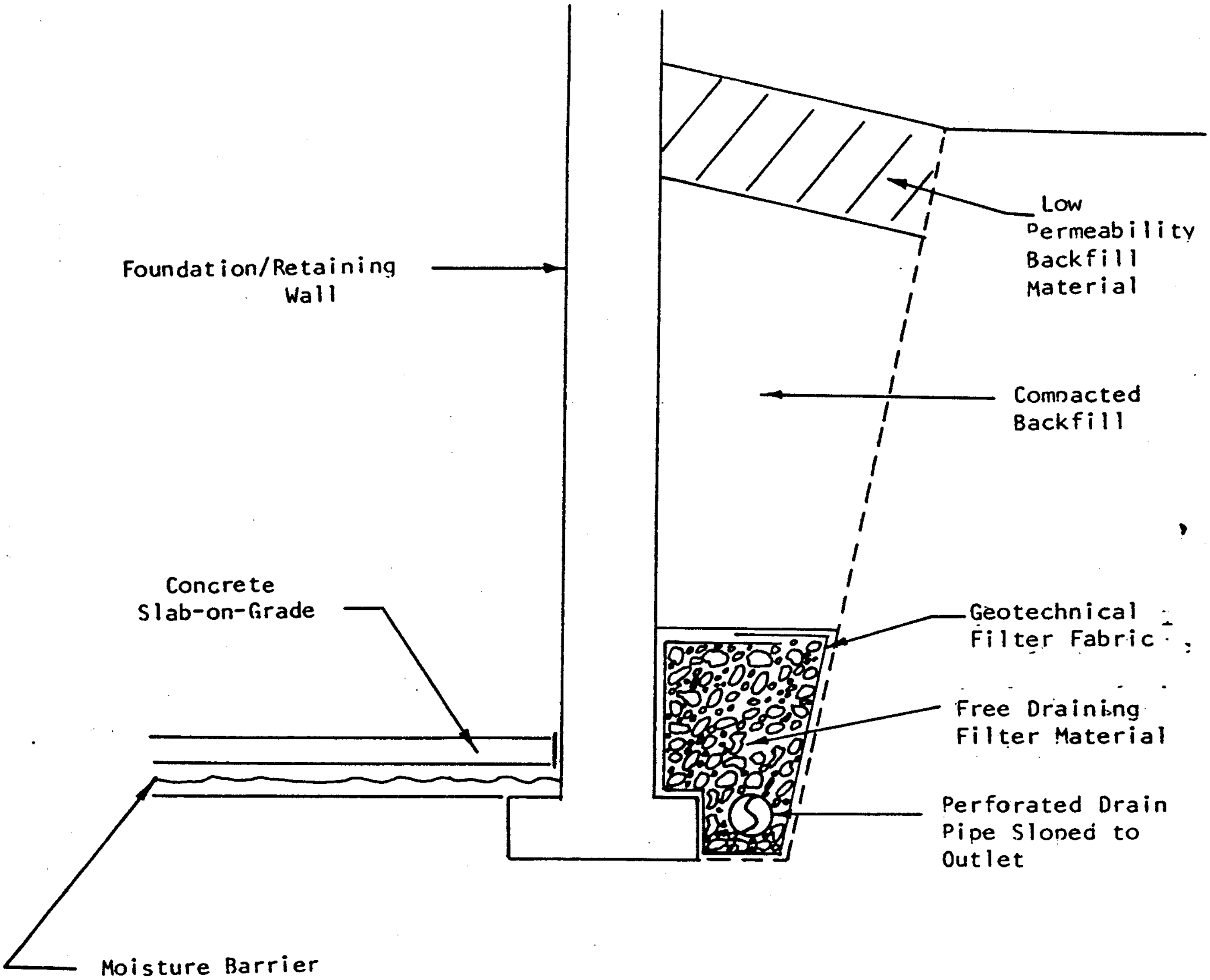
Date:	9/25/90
Figure:	4



This sketch is to show concept only.
The text of our report should be
consulted for additional information.

CONCEPTIONAL SKETCH OF FOUNDATION DRAIN SYSTEM

Lambert and Associates

Project No.:	M90140GE
Date:	9/25/90
Figure:	5

APPENDIX A

The field drilling study was performed on August 29, 199Ø. The field study consisted of logging and sampling the soils encountered in six (6) test borings in the proposed building site and three (3) test borings in the proposed parking area. The approximate locations of the test borings is shown on Figure 2. The logs of the soils encountered in the test borings are presented on Figures A2 through A11.

The test borings were logged by Lambert and Associates and samples of significant soil types were obtained. The samples were obtained from the test borings using a Modified California Barrel sampler and bulk disturbed samples were obtained. Penetration blow counts were determined using a 14Ø pound hammer free falling 3Ø inches. The blow counts are presented on the logs of the test borings such as 5/3 where 5Ø blows with the hammer were required to drive the sampler 3 inches.

The engineering field description and major soil classification are based on our interpretation of the materials encountered and are prepared according to the Unified Soil Classification System, ASTM D2488. Since the description and classification which appear on the test boring logs are intended to be that which most accurately describes a given interval of the test boring (frequently an interval of several feet) discrepancies do occur in the Unified Soil Classification System nomenclature between that interval and a particular sample in the interval. For example, an interval on the test boring logs may be identified as a silty sand (SM) while one sample taken within the interval may have individually been identified as a sandy silt (ML). This discrepancy is frequently allowed to remain to emphasize the occurrence of local textural variations in the interval.

The stratification lines presented on the logs are intended to present our interpretation of the subsurface conditions encountered in the test boring. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

KEY TO
LOG OF TEST BORING

Date Drilled ____________ **Field Engineer** ____________ **Boring Number** ____________

Location ______________________________ **Elevation** ____________

Diameter ____________ **Total Depth** ____________ **Water Table** ____________

Symbol	Depth	Sample Type	Sample N°	Soil Description	Laboratory Test Results
				Sand,silty,medium dense,moist,tan, (SM) Unified Soil Classification	Notes in this column indicate tests performed and test results if not plotted.
				Indicates Bulk Bag Sample	DD: Indicates dry density in pounds per cubic foot
	5	C		Indicates Drive Sample Indicates Sampler Type: C - Modified California St - Standard Split Spoon H - Hand Sampler	MC: Indicates moisture content as percent of dry unit weight LL: Indicates Liquid Limit
			7/12	Indicates seven blows required to drive the sampler twelve inches with a hammer that weighs one hundred forty pounds and is dropped thirty inches.	PL: Indicates Plastic Limit PI: Indicates Plasticity Index
	10			BOUNCE: Indicates no further penetration occurred with additional blows with the hammer	
				NR: Indicates no sample recovered	
	15			CAVED: Indicates depth the test boring caved after drilling	
▽				Indicates the location of free subsurface water when measured	
				CLAY NOTE: Symbols are often used only to help visually identify the described information presented on the log.	
				SILT	
	20			SAND	
				GRAVEL	
				CLAYSTONE	
				SANDSTONE	
	25				

Project Name First Presbyterian Church **Project Number** M90140GE **Figure** A1

Lambert and Associates

LOG OF TEST BORING

Date Drilled 8/29/90 **Field Engineer** Frank **Boring Number** 1

Location **Elevation**

Diameter 4 inches **Total Depth** 27 feet **Water Table** 9 feet

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
				Sand,clayey,soft,slightly moist,light grey-brown (SC)	
		ST	15/6 20/6 32/6	Clay,sandy,hard,slightly moist,grey to tan (CL) chemical deposits	
	5				
▽		ST	20/6 33/6		Swell Consolidation Test MC: 7.3% DD: 106.0 pcf Direct Shear Strength Test MC: 10.3% DD: 125.0 pcf
	10			More stiff at 10 1/2 feet More moist with depth	
		ST	12/6 12/6	Sand and gravel,slightly clayey, medium dense,wet,brown-grey (SC-GC)	
	15				
		ST	4/6 6/6		
	20			More stiff at 22 feet Stiff at 22 1/2 feet	
				Formational material,shale,hard,grey Mancos Formation	
	25				

Project Name First Presbyterian Church **Project Number** M90140GE **Figure** A2

LOG OF TEST BORING
(continued)

Date Drilled 8/29/90 Field Engineer Frank Boring Number 1 continued

Location Elevation

Diameter 4 inches Total Depth 27 feet Water Table 9 feet

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
		ST	52/6	Formational material, shale, hard, grey, Mancos Formation Continued	
	30			Bottom of test boring 1 at 27 feet	
	35				
	40				
	45				
	50				

Project Name First Presbyterian Church Project Number M90140GE Figure A3

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 8/29/90 Field Engineer Frank Boring Number 2

Location Elevation

Diameter 4 inches Total Depth 24 feet Water Table 9 feet

Water	Depth	Sample Type	N	Soil Description	Laboratory Test Results
				Sand, clayey, loose, slightly moist, light grey-brown (SC)	
				Clay, sandy, medium stiff, slightly moist, grey (CL)	
				Some cobbles	
				More stiff with depth	
	5				
∇					
	10			More soft at 10 feet	
	15				
				More stiff at 16 feet	
				More stiff at 18 feet	
	20				
				Formational material, shale, hard, brown to grey, Mancos Formation	
				Bottom of test boring 2 at 24 feet	
	25				

Project Name First Presbyterian Church Project Number M90140GE Figure A4

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Drilled 8/29/90 Field Engineer Frank Boring Number 3

tion ______________________ Elevation ______________________

eter 4 inches Total Depth 24 feet Water Table 10 1/2 feet

	Sample Type	Sample N	Soil Description	Laboratory Test Results
			Sand and clay, loose, slightly moist, red-brown (SC)	
			Clay, sandy, medium dense, moist, grey (CL)	
5			More stiff at 5 1/2 feet	
10			More soft at 10 1/2 feet	
15			More stiff at 16 1/2 feet	
			More stiff at 19 feet	
0			Formationl material, shale, hard, brown to grey, Mancos Formation	
			Bottom of test boring 3 at 24 feet	

ct Name First Presbyterian Church Project Number M90140GE Figure A5

LOG OF TEST BORING

Date Drilled 8/29/90 **Field Engineer** Frank **Boring Number** 6

Location ______ **Elevation** ______

Diameter 4 inches **Total Depth** 19 1/2 feet **Water Table** 11 feet

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
		BULK		Sand and clay, loose, slightly moist, red-brown (SC)	
	5				
		BULK		Clay, sandy, silty, medium stiff, moist (CL-SC)	
	10				
∇				More soft at 11 feet	
		BULK			
	15			More stiff at 15 feet	
		BULK		Formational material, shale, hard, brown to grey, some weathering, Mancos Formation	
		C	50/5		
	20			Bottom of test boring 6 at 19 1/2 feet	
	25				

Project Name First Presbyterian Church **Project Number** M90140GE **Figure** A8

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 8/29/90 **Field Engineer** Frank **Boring Number** 7

Location ______ **Elevation** ______

Diameter 4 inches **Total Depth** 4 feet **Water Table** None encountered

Symbol	Depth (ft)	Sample Type	Sample N	Soil Description	Laboratory Test Results
		Bulk		Sand and clay, loose, slightly moist, brown (SC)	
	5			Bottom of test boring 7 at 4 feet	
	10				
	15				
	20				
	25				

Project Name First Presbyterian Church **Project Number** M90140GE **Figure** A9

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 8/29/90 **Field Engineer** Frank **Boring Number** 8

Location ______ **Elevation** ______

Diameter 4 inches **Total Depth** 4 feet **Water Table** None encountered

Symbol	Depth (ft)	Sample Type	Sample N	Soil Description	Laboratory Test Results
		Bulk		Clay and sand, soft, slightly moist, brown (CL-SC)	
	5			Bottom of test boring 8 at 4 feet	
	10				
	15				
	20				
	25				

Project Name First Presbyterian Church **Project Number** M90140GE **Figure** A10

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 8/29/90 **Field Engineer** Frank **Boring Number** 9

Location ____________ **Elevation** ____________

Diameter 4 inches **Total Depth** 4 feet **Water Table** None encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
		Bulk		Clay, sandy, loose, slightly moist, brown (CL-SC)	
	5			Bottom of test boring 9 at 4 feet	
	10				
	15				
	20				
	25				

Project Name First Presbyterian Church **Project Number** M90140GE **Figure** A10

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

APPENDIX B

The laboratory study consisted of performing:

- Moisture content and dry density tests,
- Direct shear strength tests,
- Swell consolidation tests,
- Sieve analysis tests,
- California bearing ratio tests,
- Moisture relationship tests, and
- Chemical Tests.

It should be noted that samples obtained using a drive type sleeve sampler may experience some disturbance during the sampling operations. The test results obtained using these samples are used only as indicators of the in situ soil characteristics.

TESTING

Moisture Content and Dry Density

Moisture content and dry density were determined for each sample tested of the samples obtained. The moisture content was determined according to ASTM Test Method D2216 by obtaining the moisture sample from the drive sleeve. The dry density of the sample was determined by using the wet weight of the entire sample tested. The results of the moisture and dry density determinations are presented on the logs of test borings, Figures A2 through A11.

Swell Tests

Loaded swell tests were performed on samples obtained during the field study. These tests are performed in general accordance with ASTM Test Method D2435 to the extent that the same equipment and sample dimensions used for consolidation testing are used for the determination of expansion. A sample is subjected to a static surcharge, water is introduced to produce saturation, and volume change is measured as in ASTM Test Method D2435. Results are reported as percent change in sample height.

Consolidation Tests

One dimensional consolidation properties of drive samples were evaluated according to the provisions of ASTM Test Method D2435. Water was added in all cases during the test. Exclusive of special readings during consolidation rate tests, readings during an increment of load were taken regularly until the change in sample height was less than Ø.ØØ1 inch over a two hour period. The results of the swell-consolidation load test are summarized on Figure B1, swell-consolidation tests.

It should be noted that the graphic presentation of consolidation data is a presentation of volume change with change in axial load. As a result, both expansion and consolidation can be illustrated.

Direct Shear Strength Tests

Direct shear strength properties of samples were evaluated in general accordance with testing procedures defined by ASTM Test Method D3Ø8Ø. The direct shear strength test was performed on sample obtained from test boring 1 at a depth of nine (9) feet and test boring 5 at a depth of five (5) feet. Based on the results of the direct shear strength tests an internal angle of friction of 2Ø degrees and a cohesion of 225 pounds per square foot were used in our analysis.

Sieve Analysis Tests

Sieve analysis tests were conducted on selected samples of the material obtained during our field study. The sieve analysis tests were conducted in general accordance with ASTM Test Method D422. The results of the sieve analysis tests are presented on Figure B2.

California Bearing Ratio Tests

California bearing ratio tests were conducted on select soil samples obtained during our field study. The California Bearing Ratio tests were conducted in accordance with ASTM Test Method D1883. The results of the California Bearing Ratio tests are presented on Figure B3.

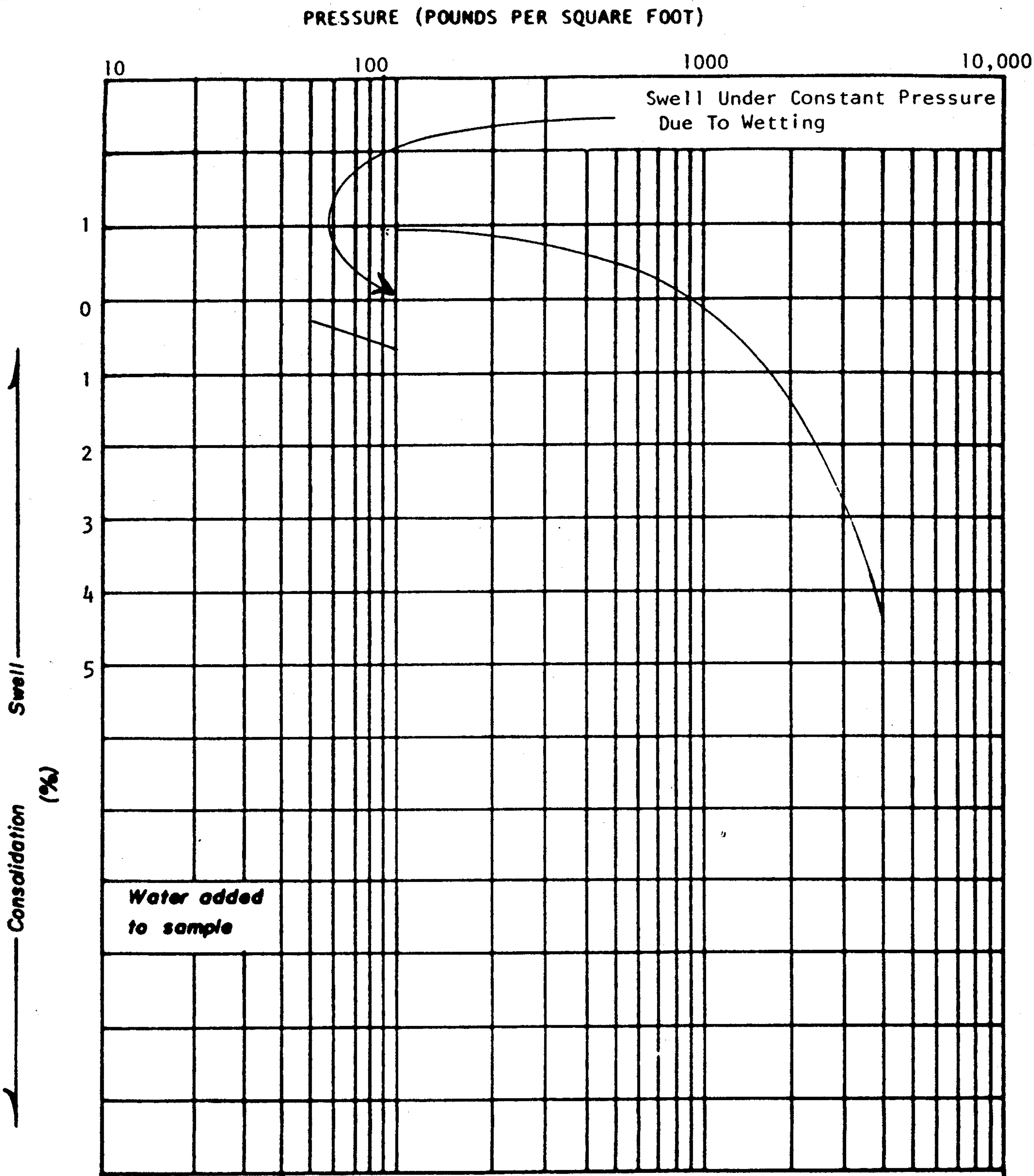
Moisture-Density Relationship Tests

Moisture-density relationship tests were conducted on select soil subgrade samples obtained during our field study. The moisture-density relationship tests were conducted in accordance with ASTM Test Method D698. The results of the moisture-density relationship tests are presented on Figure B3.

Chemical Tests

Chemical tests for water soluble sulfates, pH, and total dissolved salts were performed by Grand Junction Laboratories on select samples obtained during the field study. The results of the chemical tests are tabulated below.

Test Boring	4
Depth	1 to 3 feet
pH	8.4
Water Soluble Sulfate	Ø.Ø15%
Total Dissolved Salts	Ø.Ø39%



Boring No. 1 Depth 9 feet	SUMMARY OF TEST RESULTS				
	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
Initial	7.3	106.0	1.0	1.94	800-900
Final	18.5	113.0	.957	1.94	
Soil Description	Clay, brown				

SWELL - CONSOLIDATION TEST	Project No.:	M90140GE
Lambert and Associates	Date:	9/25/90
	Figure:	B1

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

TEST RESULTS

PROJECT First Presbyterian Church PROJECT NO. M90140GE DATE 9/13/90

LOCATION Grand Junction, CO SOURCE TH-1 @ 9 Feet

SAMPLE NO. 3773 SPECIFICATION*

SIEVE ANALYSIS

U. S. STD. SIEVE SIZE	CUMULATIVE PERCENT PASSING
-200	75

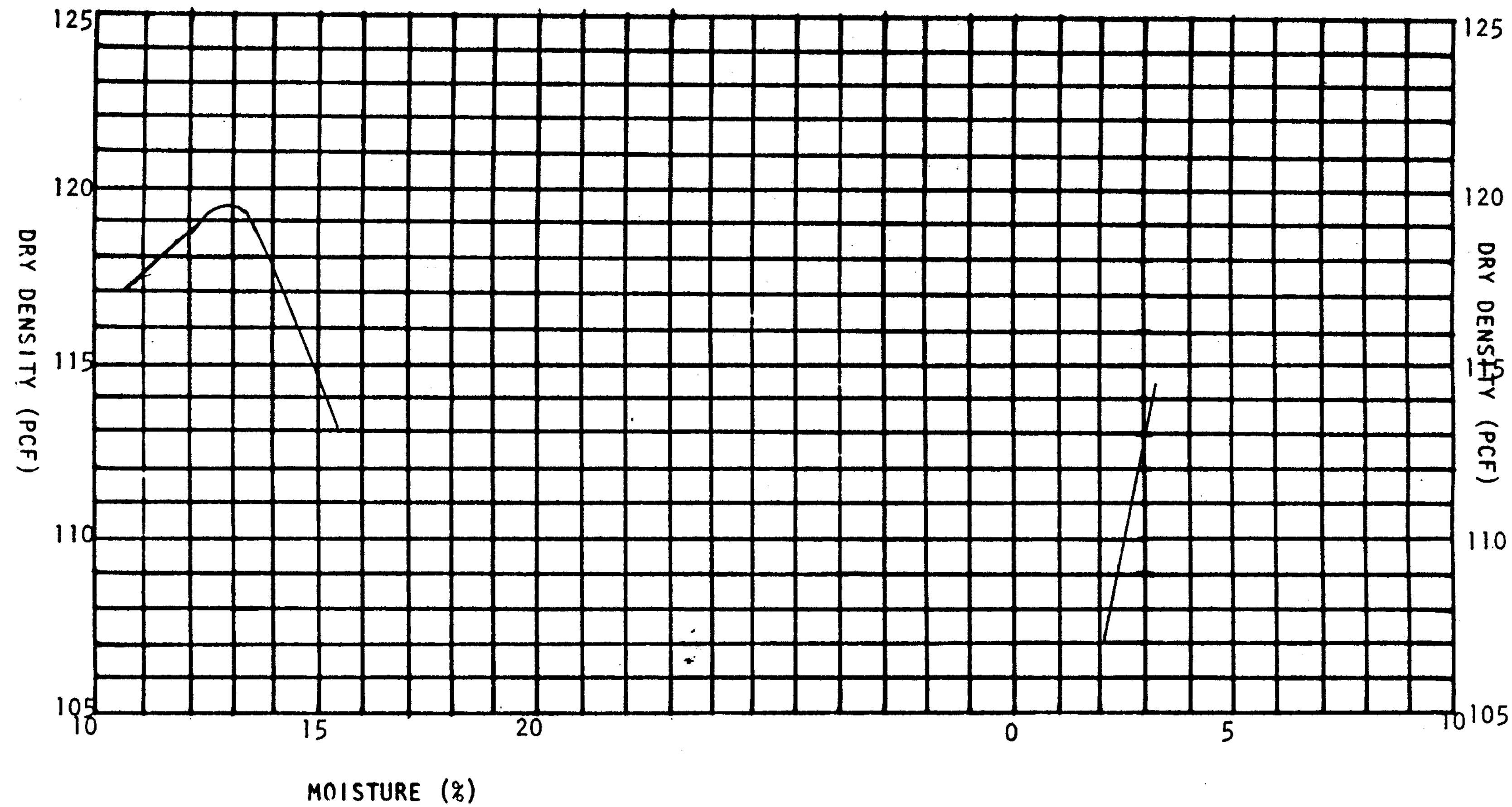
Moisture Content: 7.3%

Sampled On: 8/29/90

Clay, slightly sandy, brown

Figure B2

*It is our understanding that the noted specification is the project specification.



MOISTURE-DENSITY RELATIONSHIP
ASTM D698

MAXIMUM DRY DENSITY = 119.5
OPTIMUM MOISTURE CONTENT = 13.0

SAMPLE DESCRIPTION: Clay, sandy, brown

SAMPLE LOCATION: TB-7 at 1 to 4 feet

CALIFORNIA BEARING RATIO
ASTM D1883 (Soaked 96 hours)

METHOD OF COMPACTION: ASTM D698 Method B

PRE-SOAK		AFTER SOAK		
DRY DENSITY (PCF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	SWELL (%)
107.0	13.0	107.0	18.9	0.0
113.3	15.3	110.5	17.2	0.2
114.1	15.3	113.9	16.7	0.2

CBR at 95 percent relative compaction = 3

Lambert and Associates

Project No.: M90140GE
Date: 9/25/90
Figure: B3

TO:
THE METROPOLITAN PLANNING ORGANIZATION
for the Grand Junction urbanized area

COMMENTS AND RECOMMENDATIONS
regarding the
MAJOR ARTERIAL CORRIDOR STUDY
and the
NORTHEAST AREA TRANSPORTATION STUDY

Submitted by
Kenneth L. Etter
P. O. Box 1653
Grand Junction, CO 81502

October 1, 1990

27-1/2 Road - The only local traffic route from the Horizon Drive Area to the East, has consisted of the awkward route: from Horizon Drive to G Road, to 27-1/2 Road, to F Road. Providing a smoother traffic flow is sorely needed.

Not only is this route awkward but other problems obstruct this route (as it currently exists) from being upgraded for a planned major traffic route. Those include: numerous driveways from single family residences on 27-1/2 Road (between F and G Roads); a ridge and a gulley which obstruct vision of the road ahead, from both directions on 27-1/2 Road; Both 27-1/2 and G Roads, at their intersection, are prescriptive easements. Widening and improving this intersection would require land acquisitons and cause severe damage to the land owner at the Southwest corner of this intersection. Numerous giant trees would have to be destroyed. The expense of compensating the landowner would be substanial.

Copy

G Road - G Road would extend over 16 miles, if entirely constructed from its Eastern most limit (at "projected" 38-3/4 Road) near Palisade, to its Western most limit (at "projcted" 22-1/2 Road). Currently, 11 miles have been constructed, (the Easterly most 6 miles and the Westerly most 5 miles.)

The Western portion of G Road extends East, past 27-1/2 Road (and provides an adequate traffic route to the West from Horizon Drive.) The Westerly portion of G Road cannot be extended, in a straight line to the East, very far beyond its current extent. A canal, canal road, I-70 prohibit the straight line extension of G Road. The "G Road" route must shift to the South if this route is extended further East.

Connector Road: G to F-3/4 - Because of development in the area, new traffic routes from Horizon Drive to the East, will have to use either G Road (which can extend only a short distance due East) or F-3/4 Road. A new road which would connect G Road to F-3/4 Road, and cross (or intersect) Horizon Drive at the desired 90 degree angle, is possible. Any such new road, perpendicular to Horizon Drive, would have to be located no further East than the intersection of Horizon Drive and G Road; and no further West than the intersection of Horizon Drive and F-3/4 (Cortland Ave.)

If this road is located at its Eastern most possibility, it would cross 27-1/2 Road (at about a 45 degree angle) before merging with F-3/4 Road (also at about a 45 degree angle). This particular configuration of roads would allow a smooth traffic flow, from Horizon Drive, to both the East (on F-3/4 Road) and to the South on 27-1/2 Road.

Situating the connector road further to the West would result in its intersection with F-3/4 Road before the connector crossed 27-1/2 Road. Traffic moving from the Horizon Drive Area, to the South would have a 45 degree angle turn to the left, then a 90 degree turn to the right, rather than just one 45 degree angle turn to the right. See Exhibit "E".

F-3/4 Road (Cortland Ave.) - This road should become a major artery East of Horizon Drive. However, this road, alone will not provide an improved traffic flow from the Horizon Drive (North of G Road) to the East. The intersection of F-3/4 and Horizon Drive requires traffic to travel too far West - out of the way. Prudent road planning requires the use of the previously descibed connector road, with F-3/4.

Note worthy portions of this road have been constructed: between 27-1/2 and 28 Roads; 30 and 30-1/2 Roads; 30-3/4 and 31-1/2 Roads; and more than one hundred feet East of Horizon Drive (extending towards 27-1/2 Road). See Exhibit "E". F-3/4 Road shoud be extended from 28 Road to 28-1/4 Road in the near future (and further East, later).

Residential development exists on both sides of F-3/4 Road between 27-1/2 Road and 28 Road. Prudent planning has prevented drive ways from being constructed directly off of F-3/4. This planning measure should be continued on other portions of roads which may be a part of any subsequent major traffic arteries.

REVIEW SHEET SUMMARY

FILE NO. #43-90 **TITLE HEADING:** Revised Final Plan

ACTIVITY: Revised Final Plan / First Presbyterian Church

PETITIONER: First Presbyterian Church

RESPONSE NECESSARY by NOV 2 1990

REPRESENTATIVE: Self

LOCATION: Northeast corner Cortland Avenue & 27 1/2 Road

PHASE: Final **ACRES:** 8.97 acres

PETITIONER'S ADDRESS: 622 White Ave, Grand Junction

ENGINEER: Reck & Associates, 9605 W 49th St, Wheatridge, CO 80033

STAFF REPRESENTATIVE: David Thornton

NOTE: WRITTEN RESPONSE BY THE PETITIONER TO THE REVIEW COMMENTS IS REQUIRED A MINIMUM OF 48 HOURS PRIOR TO THE FIRST SCHEDULED PUBLIC HEARING.

CITY POLICE DEPARTMENT **10/15/90**
J.E. Hall 244-3577

Do not anticipate this would have nay adverse impact on the Police Department.

CITY FIRE DEPARTMENT **10/10/90**
George Bennett 244-1400

11-6-90 No Problems

The following items need to be reviewed by our department:

1. Building plans - to accomplish a required fire flow.

2. Fire alarm system plans review.

Access and fire hydrant and line size appear to be adequate at this time.

WALKER FIELD AIRPORT **10/16/90**
Jeff Wendland, Airport Manager 244-9120

No Problem

No FAA or Airport Authority prohibition for this use.

Request standard avigation easement be executed and forwarded to Airport Authority. We're available to assist.

Suggest noise mitigation be considered in design and construction. We're available to assist.

CITY ENGINEER **10/19/90**
Don Newton 244-1559

No Problems 11-6-90

Limits of construction of new street improvement on 27 1/2 Road and Cortland Avenue are not clear. Will need to submit detailed drawings for public street improvements to be constructed, including profiles and cross-sections. Submit pavement design calculations and typical street section for Cortland Avenue.

CITY ENGINEER **10/19/90**
Don Newton 244-1559

11-6-90
NO Problems

All fire lanes should be designed for truck loading. Fire lane should also be signed to prevent parking. Stop signs should be installed at all exits on to public streets.

Storm runoff volume from the site is over estimated using peak Q for entire duration of the storm. How does this affect the size of outlet from the detention basin? How was Q from Apple Crest Subdivision determined? Show calculations. Drainage inlet from detention basin should allow direct access to the top of the outlet pipe for maintenance purposes. What are "concrete collars" shown at bends in storm drain pipe crossing 27 1/2 Road? Storm runoff from Apple Crest appears to be directed across the parking lot surface. Recommend that a concrete drainage pan or pipe be installed to prevent deterioration of the parking lot.

What street pavement section was used for estimating quantities in the improvements agreement? The estimate for streets appears too low.

Street lights, storm sewer facilities, and construction administration need to be added to the improvements agreement.

A drainage easement should be dedicated along the alignment of the drainage from Apple Crest Subdivision.

UTE WATER DISTRICT **10/10/90**
Gary Matthews 242-7491

11-6-90
NO Problems

THIS PROJECT WILL BE SUPPLIED WITH UTE WATER.

Ute Water has a 18" main on the north side of Cortland Avenue and on the east side of 27 1/2 Road.

The fireline system could adequately be supplied with <u>ONE</u> connection to the 18" main line.

If installed as designed, it would require three leak detectors plus a minimum charge per month each detector. (See Sheet Attached)

It's not good practice to loop two leak detectors together.

Ute Water has no interest in a easement on private property. Ute's obligation would be to the property line.

POLICIES AND FEES IN EFFECT AT THE TIME OF APPLICATION WILL APPLY.

***** See Attachment "A"**

U.S. WEST **10/5/90**
Leon Peach

No comments at this time.

PUBLIC SERVICE **10/8/90**
Carl Barnkow 244-2790

GAS: No objections to revised final plan. Public Service will need to remove existing service to house prior to taking place.

ELECTRIC: No objections.

NO Problem

GRAND VALLEY WATER USERS 10/17/90
G. W. Klapwyk 242-5065

The discharging of up to 19.5 CFS of water into the channel located off this property, as per drainage plan, will undoubtedly cause downstream problems along the channel clear to or beyond 27 Road. While the channel at its upper end where the proposed discharge would enter is of sufficient size to carry the flow, the same cannot be said for it some 1/4 +/- mile downstream and particularly at crossings along the channel.

Such channel has historically collected a modest amount of seepage water and conveyed modest flows of return-flow and storm run-off water, but not of the quantity proposed herein. In addition, the Ptarmigan development downstream has addressed the matter of this channel concerning their activity and it is unknown if their considerations include the flow proposed by this development. We cannot be specific about the channel's flow capabilities, other than to say that historic flows have been much less than that herein proposed. It seems the entire matter of what is being forced on this channel by various developments should be looked into comprehensively over its entire route in an effort to avoid serious downstream problems. At this time the Association must oppose the change of historic facility sizing at the southwest corner of this development until it can be determined that such change will no create hardship, damage, and liability for all concerned in one way or another.

COMMUNITY DEVELOPMENT DEPARTMENT 10/19/90
David Thornton 244-1447

1. Signage exceeds code by six square feet. The Development Code allows 24 square feet; a 30 square foot sign is being proposed. Staff has no problem with the proposed sign since only one sign is being proposed whereas the code allows one 24 square foot sign per street frontage for a total of 48 square feet of signage. Height and content of sign conforms to Code. A separate sign permit is required for the sign.

2. We need to know size of plantings for Phase I Landscaping. The minimum allowable plant size for new installations shall be 1 1/2 inch caliper for deciduous, six feet tall for evergreens, and five-gallon size for shrubs.

3. Number of parking spaces meets Code for one space per three persons designed seating capacity.

4. Screening needs to be addressed along northern and northeast boundary of property near ballfield, picnic area, tennis courts.

5. Landscaping plan for Phase II will need to be approved by our office prior to Phase II construction.

6. All parking area/security illumination shall be arranged so as to confine direct light beams to the lighted property and away from nearby residential properties and the vision of passing motorists.

7. What is the proposed building height? The zone allows a maximum of 32 feet; although, this height limitation does not apply to church spires, belfries, cupolas, etc.

8. Existing house on property. Will this be removed? Phase I? Phase II?

COMMUNITY DEVELOPMENT - continued **10/19/90**
David Thornton 244-1447

9. All related documents must be recorded before a Planning Clearance will be granted (ie: Site Plan, Improvements Guarantee, Improvements Agreement, Avigation Easement, Utility Easement Deed, Quit Claim Deed for additional Right-of-Way, etc.)

10. All Review Agency comments must be addressed and the written response to them needs to be in our office within 48 hours of Planning Commission Hearing on November 6, 1990.

11. Petitioner is responsible for all recording costs.

12. We need elevation drawings of all portions of the church building that exceed 32 feet in height.

13. Currently, our Development Code does not allow churches in the Airport Critical Zone. This was an oversight during the original hearing for the Conditional Use permit when the Conditional Use was granted. We have determined that there are no federal mandates that state churches are incompatible land uses in Airport Critical Zones. Therefore, a text amendment is forthcoming to remove churches from the category of incompatible uses in the Airport Critical Zone area.

BUILDING DEPARTMENT **10/22/90**
Bob Lee 244-1631

No comments.

CITY UTILITIES ENGINEER **10/23/90**
Bill Cheney 244-1590

1. E.Q.U. at 4.4 for initial installation with increase to 7.5 at buildout.

2. City water cannot be furnished to the proposed location at this time; therefore, the petitioner will have to arrange for water service from another water purveyor.

3. No other comments on sewer or water, since no new extensions for either utility are required.

Reck and Associates
Architecture, Planning,
Interior Design
9605 W. 49th Avenue
Suite 300
Wheat Ridge, Colorado 80033
(303) 431-8600

October 24, 1990

Don Newton
Public Works
City of Grand Junction
250 N. 5th St.
Grand Junction, Co 81501

RE: First Presbyterian Church Revised Final Plan

Dear Mr. Newton,

Per our telephone conversation of October 24th, we have discussed your concerns in regard to the above project with the following resolutions:

The limits of street construction will be as follows:

Improvements along 27 1/2 Road will be for the east half of this roadway along the entire length of the property including paving, curb, gutter and sidewalk. These improvements are anticipated to be accomplished at some future date to be determined by the City of Grand Junction and therefore, design of these street improvements will be accomplished in relation to the timing of construction.

Improvements along Cortland Ave. will be for the north half of this roadway along the entire length of the property including paving, curb, gutter and sidewalk. These improvements are intended to be accomplished concurrent with the first phase of building construction. Design of these street improvements will be accomplished concurrent with the development of construction documents for the first phase building and will have to be reviewed and approved by the engineering dept. prior to issuance of a building permit.

All fire lanes will be designed for truck loading in accordance with the recommendations of the soils investigation for the site. Stop signs will be installed at all exits onto public streets and fire lane signage will be provided in accordance with city requirements.

Comments regarding drainage improvements will be addressed by Rolland Engineering.

The street pavement section at Cortland Ave. and at 27 1/2 Road shall be 4" asphalt on 12" base in lieu of 3" asphalt on 6" base. The improvements estimate will be modified to reflect this.

Street lighting is to be provided at the corner of Cortland Ave. and 27 1/2 Road and at the two major access points to the site. This cost will be added to the improvements estimate (Public Service estimates these at approx. $1,000 each), as well as the cost of drainage improvements and construction administration. I will forward a revised copy of the estimate to you as soon as requirements for drainage improvements are resolved between yourself and Rolland Engineering.

A drainage easement will be dedicated along the alignment of the drainage from Apple Crest Subdivision. The size of the easement will need to be coordinated with the requirements forthcoming from your meeting with Rolland Engineering and coordinated with the utility easement along this property line.

Thank-you for your time and if you have any further comments prior to Nov. 2nd, please feel free to contact me at 1-800-273-8953.

Sincerely,

Don Watt

Reck and Associates
Don Watkins

cc Dave Thornton
   Elgin Mallory

Reck and Associates
Architecture, Planning,
Interior Design
9605 W. 49th Avenue
Suite 300
Wheat Ridge, Colorado 80033
(303) 431 8600

October 24, 1990

Bill Cheney
Department of Public Works
City of Grand Junction
250 N. 5th Street
Grand Junction, CO 81501

RE: First Presbyterian Church Revised Final Plan

Dear Mr. Cheney,

Per our telephone conversation of October 24th, we have discussed your concerns in regard to the above project with the following resolutions:

The basis for sanitary sewer tap fees for the initial phase will be 4.4 units multiplied by $750.00 per unit. Total buildout will be 7.5 units and is inclusive of the initial 4.4 units. A unit is based upon 100 seats at the sanctuary.

Water will be provided by Ute Water District.

Thank-you for your time and if you have any further comments prior to Nov. 2nd, please feel free to contact me at 1-800-273-8953.

Sincerely,

Reck and Associates
Don Watkins

cc Dave Thornton
Elgin Mallory

Reck and Associates
Architecture, Planning,
Interior Design
9605 W. 49th Avenue
Suite 300
Wheat Ridge Colorado 80033
(303) 431-8600

October 24, 1990

George Bennett
Grand Junction Fire Dept.
330 S. 6th St.
Grand Junction, CO 81501

RE: First Presbyterian Church Revised Final Plan

Dear Mr. Bennett,

Per our telephone conversation of October 24th, we have discussed your concerns in regard to the above project with the following resolutions:

The fire hydrant system will need to be designed to accomodate the required fire flow. The fire flow will be determined based upon the building construction type but is estimated to be between 2,500 - 3,000 GPM (3,750 GPM maximum). The design will be reviewed with you as it is accomplished. At this time, the Owner's civil engineer estimates that a 8" supply line will be sufficent.

The fire hydrant system will not be required to be looped through the site. Per our conversation with Gary Mathews of Ute Water District, the 18" water line in 27 1/2 Road and Cortland Ave. are the same line. The line comes down 27 1/2 Road, turns at a right angle at Cortland and thence down Cortland. Therefore, a loop would simply "cut across the corner" and would not necessasarly provide a redundancy in providing water pressure to the fire hydrant system unless the 18" line were disrupted somewhere between the loop tap points. It is understood that a "dead end" system will have to accomodate the requirements noted above.

Fire alarm system plans will be submitted for review when the system design is completed. The system should be designed in the initial phase to accomodate the full buildout.

Thank-you for your time and if you have any further comments prior to Nov. 2nd, please feel free to contact me at 1-800-273-8953.

Sincerely,

Reck and Associates
Don Watkins

cc Dave Thornton
Elgin Mallory

Reck and Associates
Architecture, Planning,
Interior Design
9605 W. 49th Avenue
Suite 300
Wheat Ridge, Colorado 80033
(303) 431-8600

RECEIVED GRAND JUNCTION
PLANNING DEPARTMENT
OCT 25 1990

October 22, 1990

Dave Thornton
City of Grand Junction
Planning Department
250 N. Fifth Street
Grand Junction, CO 81501-2643

Re: First Presbyterian Church Revised Final Plan

Dear Dave,

Per our phone conversation of the 19th the following is information you have requested:

- The maximum height of the building is indicated on the elevations (Sht. 3) as 36'- 0" above the median grade of the site. The site slopes from 100' at the southwest corner of the site to 108' at the northeast corner of the site. The median grade of the site is therefore 104' which would put the highest portion of the building at a maximum of 140' site elevation (36' bldg. hgt. + 104' site datum). It is my understanding that the maximum height allowed for this zone type is 32'- 0". We would like to maintain the 36'- 0" max. hgt. Please let me know if this will require special processing or approval other then through the Revised Final Plan.

- The site data for Phase I would be:

Building Coverage	25,720 S.F.	(8%)
Paving	64,800 S.F.	(18%)
Landscaped/Open Space	262,757 S.F.	(74%)

- Lighting shall be provided at areas indicated on the Site Plan (Sht. 1) to provide night lighting for security, safety, adequate visibility for maneuvering and to emphasize entrances, exits and hazards. Lighting shall be designed so as not to unreasonably disturb occupants of adjacent residential properties.

- Fences and walls shall be placed as indicated on the Site Plan and future plantings established so as to screen adjacent residential properties to prevent disturbance due to the maneuvering of vehicles entering and leaving the parking area.

- Minimum plant sizes shall be per the City of Grand Junction Landscape Standards and Requirements, i.e.:

Deciduous trees:	1 1/2"-inch caliper
Evergreen trees:	6 feet tall
Shrubs:	5-gallon size

If this information is acceptable to you, it can be incorporated into the submittal materials or presented in another format you may require.

Sincerely,

Don Watkins

Reck and Associates
Don Watkins

cc First Presbyterian Church - Elgin Mallory

ROLLAND ENGINEERING
518 28 ROAD SUITE B-103
GRAND JUNCTION, COLORADO 81501
(303) 243-8300

November 1, 1990

Mr. Don Newton
City Engineer
Public Works Department
250 N. 5th Street
Grand Junction, CO 81501

RE: Response to City Engineer's review comments pertaining to the proposed First Presbyterian Church of Grand Junction Drainage / Grading Plan.

Dear Don,

This letter is in regards to our meeting on Wednesday, October 31, 1990 and our telephone conversation today, Thursday, November 1, 1990 concerning the drainage plan for the proposed First Presbyterian Church site.

As a result of our discussions it was concluded that; 1) the release rate from the site would be equivalent to the historic release rate of approximately 9 cfs, 2) there would be a 5 foot drainage easement dedicated adjacent to the 10 foot utility easement along the southerly property line of the site, and 3) the location of the drainage inlet for the lower detention pond would be located in such a way to provide easy access to the top of the outlet pipe for maintenance purposes.

We appreciate your time and consideration concerning this matter.

Sincerely,

Mark D. Young

Mark D. Young
ROLLAND ENGINEERING

MDY/cfo

ROLLAND ENGINEERING
518 28 ROAD SUITE B-103
GRAND JUNCTION, COLORADO 81501
(303) 243-8300

November 1, 1990

Mr. Bill Klapwyk
Grand Valley Water Users Association
500 S. 10th
Grand Junction, CO 81501

RE: Response to Grand Valley Water Users Associations review comments pertaining to the proposed First Presbyterian Church of Grand Junction Drainage/Granding Plan.

Dear Bill,

Upon review of your comments concerning the amount of proposed discharge of up to 19.5 cubic feet per second (cfs) into the channel located off the church property, and realizing the potential impact of this discharge downstream, we have revised the initial Drainage Plan for the First Presbyterian Church to address your concerns.

Per our telephone conversation Wednesday, October 31, 1990 it was determined that if a release rate equivalent to the historic release rate of approximately 9 cfs could be achieved that that would be acceptable to the Grand Valley Water Users Association.

Thus, to accommodate Grand Valley Water Users Associations request, the drainage system for the proposed First Presbyterian Church has been designed to release approximately 9 cfs.

We appreciate your time and consideration concerning this matter.

Sincerely,

Mark D. Young

Mark D. Young
ROLLAND ENGINEERING

MDY/cfo

Reck and Associates
Architecture, Planning,
Interior Design
9605 W. 49th Avenue
Suite 300
Wheat Ridge, Colorado 80033
(303) 431 8600

November 2, 1990

Dave Thornton
Community Development
City of Grand Junction
250 N. 5th Street
Grand Junction, CO 81501

Re: First Presbyterian Church Revised Final Plan

Dear Dave,

Rolland Engineering and myself have addressed the comments and concerns of the following agencies and correspondence with them is attached:

City Fire Department
Walker Field Airport
City Engineer
Grand Valley Water Users
Ute Water District
City Utilities Engineer

The following agencies did not have concerns to be addressed:

City Police Department
U.S. West
Public Service
Building Department

The following are in response to Community Development's comments:

Minimum plant sizes shall be per the City of Grand Junction Landscape Standards and Requirements, i.e.:

Deciduous trees:	1 1/2"-inch caliper
Evergreen trees:	6 feet tall
Shrubs:	5-gallon size

Fences and walls shall be placed as indicated on the Site Plan and future plantings established so as to screen adjacent residential properties to prevent disturbance due to the maneuvering of vehicles entering and leaving the parking area and due to activities associated with the picnic area, tennis court and ballfield. This landscaping scheme will be developed and reviewed with the City of Grand Junction concurrent with the development future improvements.

Lighting shall be provided at areas indicated on the Site Plan (Sht. 1) to provide night lighting for security, safety, adequate visibility for maneuvering and to emphasize entrances, exits and hazards. Lighting shall be designed so as to confine direct light beams to the lighted property and away from nearby residential properties and the vision of passing motorists.

The maximum building height shall be 32'-0" with the exception of towers and spires. (See attached sketch).

The existing house will remain with the Initial Improvements Phase and be removed with the development of Future Improvements.

If you have any further comments please contact either myself or Tom Reck. I will be attending the Planning Commission Hearing scheduled for Tuesday, November 6th at 7:30 P.M. at City Hall.

Sincerely,

Reck and Associates
Don Watkins

cc Elgin Mallory

FF — ACTION SHEET

Original Do NOT Remove From Office

FINAL

Already Received C.U. for CHURCH see File # 15-86

ACRES 8.97

UNITS

DENSITY

FILE NUMBER #43 90

ZONE RSF-4

TAX SCHEDULE # 2945-011-00-100

ACTIVITY Revised Final Plan - Major Changes

PHASE Final

COMMON LOCATION N.E. Corner of 27 1/2 Road & Cortland Avenue

DATE SUBMITTED ____ DATE MAILED OUT ____ DATE POSTED ____

____ DAY REVIEW PERIOD RETURN BY ____

OPEN SPACE DEDICATION (acreage) N/A OPEN SPACE FEE REQUIRED $ N/A PAID RECEIPT # N/A

RECORDING FEE REQUIRED $ ____ PAID (Date) ____ DATE RECORDED ____

REVIEW AGENCIES

A B C D E F G H I J K L M N O P Q R S T U V W X Y Z AA BB CC DD EE FF GG

- ● Planning Department
- ● City Engineer
- ○ Transportation Engineer
- ○ City Parks/Recreation
- ● City Fire Department
- ● City Police Department
- ○ County Planning
- ○ County Engineer
- ○ County Health
- ○ Floodplain Administration
- ○ G.J. Dept. of Energy
- ● Walker Field
- ○ School District
- ● Irrigation
- ● Drainage
- ● Water (Ute, Clifton)
- ○ Sewer Dist. (FV, CGV, OM)
- ● U.S. West
- ● Public Service (2 sets)
- ○ State Highway Department
- ○ State Geological
- ○ State Health Department
- ● City Property Agent
- ● City Utilities Engineer
- ● City Attorney
- ● Building Department
- ○ DDA
- ● GJPC (7 packets)
- ○ CIC (11 packets)
- ○ Other

TOTALS

BOARDS	DATE	
PC	11/6/90	Approved subject to review sheet summary and specifically to allow one 30 sq ft. sign. Appeal made found invalid -- PC decision final

STAFF

APPLICATION FEE REQUIREMENTS

Final = 225.00 + 125.00 Acreage fees = TOTAL $350.00

Grand Junction Planning

* ORIGINAL FORMS WITH SIGNATURES INCLUDE IN PLANNING DEPT. PACKET *

MEMORANDUM

File # 43-90

November 6, 1990

To: David Thornton
From: Don Newton

Re: First Presbyterian Church

This is to inform you that the project Architect and Engineer have satisfactorily addressed my comments and concerns on this project.

An additional storm runoff detention basin will be constructed at the south east corner of the property to control the runoff from Crown Heights Sub. This will limit the release of runoff from all developments to 9 cfs which is the historic undeveloped rate from a 10 year storm

Half street improvements on Cortland Ave. will be constructed with development of the church. An acceptable form credit will be provided for the future improvement of 27 1/2 Road.

KENNETH L. ETTER
P. O. Box 1653
Grand Junction, Colorado 81502

November 7, 1990

To The City of Grand Junction

Regarding Planning Commission Item #43-90

The land owner has petitioned the city for a revised final plan for the construction of a church and school. Be advised that the original conditional use permit was apparently granted in error. The subject land is in the critical zone for Walker Field runway 4-22. The original conditional use permit, granted in 1986, violated zoning criteria. A church is an incompatible use in an airport critical zone. The request for a revised final plan continues this incompatible use.

In additon, a major traffic corridor from the intersection of G Road and Horizon Drive, to about 2740 F-3/4 Road, may be essential for an adequate traffic system. This route would cross the center the subject tract of land. The city's approval of planning commission item #25-90 eliminates any other possible route for such a road (which would be perpendicular to Horizon Drive and connect G Road to F-3/4 Road).

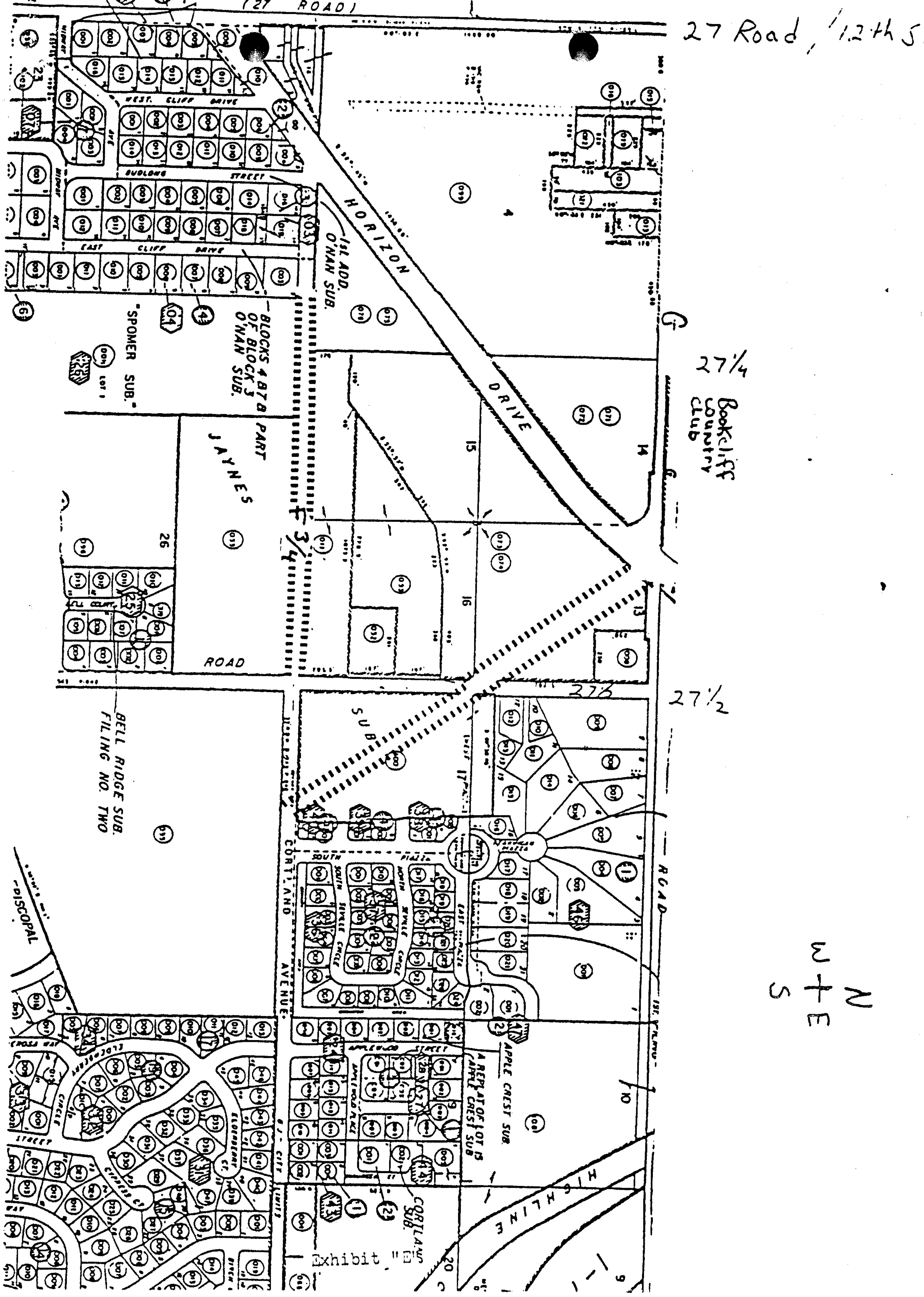
Sincerely,

Ken Etter

Kenneth Lamar Etter

RECEIVED GRAND JUNCTION
PLANNING DEPARTMENT

NOV 08 1990



27 Road, 12th St
(27 ROAD)
WEST CLIFF DRIVE
DUBLONG STREET
EAST CLIFF DRIVE
HORIZON DRIVE
ISL. ADD. O'NAN SUB.
BLOCKS 4, 8 T B PART OF BLOCK 3 O'NAN SUB.
"SPOMER SUB."
JAYNES
F 3/4
ROAD
G
27 1/4
Bookcliff Country Club
27 1/2
BELL RIDGE SUB. FILING NO. TWO
CORTLAND AVENUE
SOUTH PLAZA
EAST PLAZA
APPLE CREST SUB.
A REPLAT OF LOT 15 APPLE CREST SUB.
APPLEWOOD STREET
CORTLAND SUB.
HIGHLINE
ROAD
EPISCOPAL
CITY LIMITS
N
W E
S

Exhibit "E"

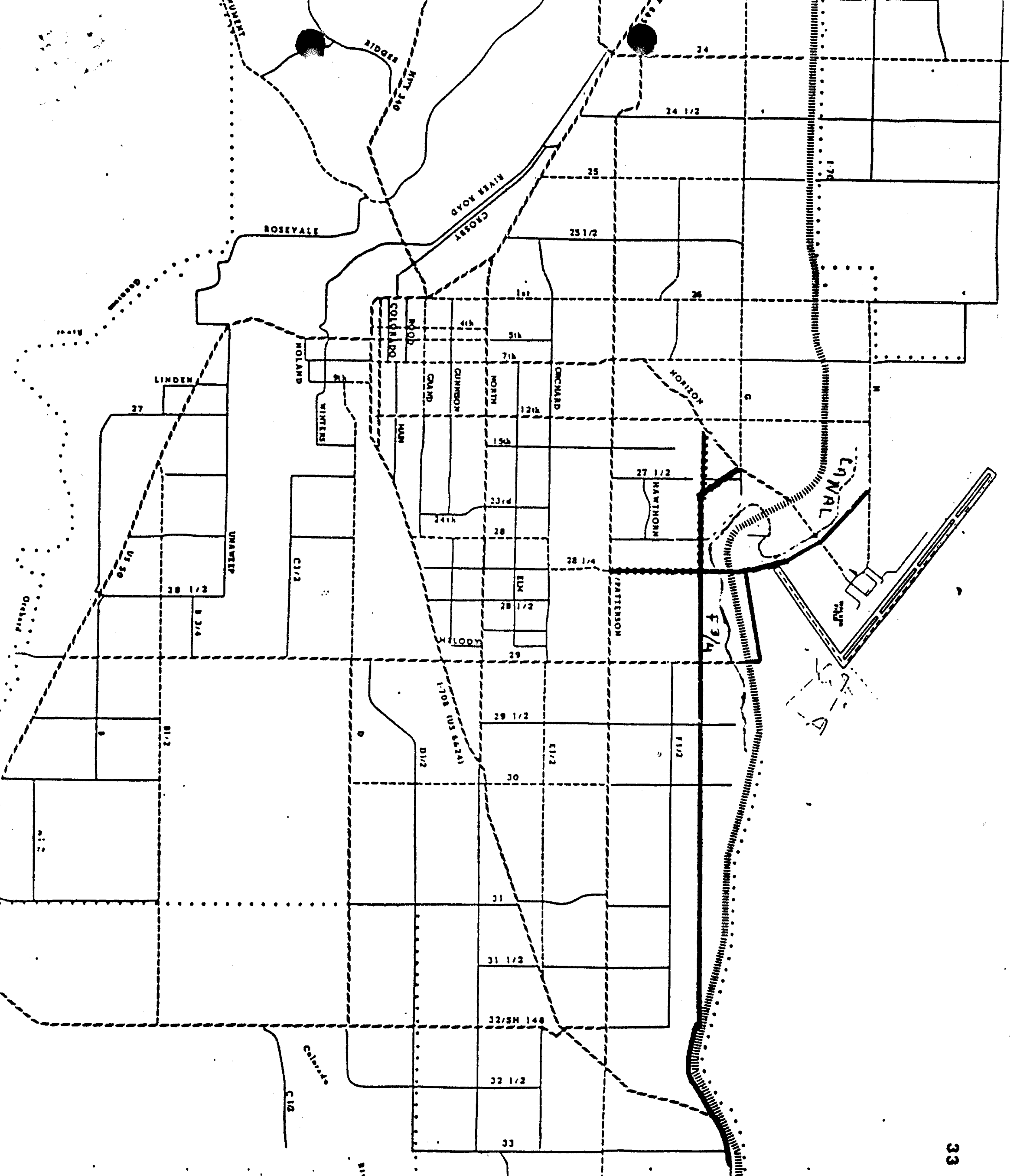

ROSEVALE
RIVER ROAD
CROSBY
1st
4th
5th
7th
12th
15th
23rd
24th
COLORADO
ROOD
NOLAND
LINDEN
WINTERS
GRAND
GUNNISON
NORTH
ORCHARD
MAIN
HORIZON
HAWTHORN
PATTERSON
ELM
MELODY
UNAWEEP
US 50
CANAL
F 3/4
G
24
24 1/2
25
25 1/2
26
27
27 1/2
28
28 1/4
28 1/2
29
29 1/2
30
31
31 1/2
32/SH 141
32 1/2
33
34
C 1/2
D
D 1/2
E 1/2
F 1/2

Exhibit "F"

Marty

J: see second page

Please return to file # 43-90

KP ✓ KM ✓

DT ✓

BP ✓

LWV

MEMORANDUM

TO: DAN WILSON, CITY ATTORNEY

FROM: JOHN SHAVER, ASSISTANT CITY ATTORNEY JS

DATE: 13 NOVEMBER 1990

RE: PLANNING COMMISSION ITEM 43-90

On November 6, 1990 the planning commission approved a final plan, as submitted, for the development of the First Presbyterian Church at the northeast corner of 27 1/2 Road and Cortland Avenue. As you will recall this project was approved in 1986 but the church has substantially modified the original plan and has come back for final approval.

The project has presented some difficulties for the Planning staff because the church property is located in an airport critical zone. The critical zone designation is imposed by the City's development code and defines the church use as incompatible therein. Since the project has been previously approved and any recission of that approval status or further delay may subject the City to liability and the fact that the only incompatibility of this type of use in a critical zone is because of noise, the Planning staff, upon recommendation of the assistant city attorney suggested that the plan be forwarded on to Planning Commission and that the project be considered. The assistant city attorney also recommended that a text amendment should be prepared or the airport compatibility section of the Code should be rewritten to properly reflect appropriate and inappropriate uses in the airport area.

At the hearing on the 6th, Mr. Kenneth Etter spoke against the location of the church on this parcel stating that it was a necessary piece of ground for his proposed traffic corridor. Mr. Etter insisted that a traffic corridor is essential and that the approval of the final plan is "illegal". It was explained to Mr. Etter that the appropriate time to remonstrate was at the time of approval in 1986.

On November 8, 1990 Etter filed the attached letter and maps with the Planning Department. The letter is intended to be an appeal of the Planning Commission decision. The letter, as you can see, does not state that it is an appeal and it also refers to another item previously considered by the Planning Commission and the Council. The question that Mr. Etter's letter raises is whether he or the persons he represents have standing to challenge the Commission's decision.

The Code section is not clear. The pertinent section 2-2-2 C 3. states that "decisions ... may be appealed to the Governing Body by any person who is given standing by this Code". The Code then fails to further define who is given standing.

The case law on standing is clear as to what a person must allege in order to have standing before the Courts. Standing is one of the fundamental prerequisites to a Court hearing a case.

> Standing is an aspect of justiciability as that term relates to the case or controversy requirement of Article II section 2 of the United States Constitution. Hinkson v. Pfleiderer, 729 P.2d 697, 1984.
>
> Standing is not conferred on the merits of the claim but on the nature of the injury.
>
> Standing problems are analyzed in terms of whether a party alleges that the challenged action has caused injury in fact, economic or otherwise, and whether the interest sought to be protected by the complainant is arguably within the zone of interest to be protected ... party must demonstrate by facts alleged that he himself has been adversely affected or those he represents have been injured in fact. Citizens Concerned for Separation of Church and State v. City and County of Denver, 628 F.2d 1289, cert. den. 452 U.S. 963, 1980.

If Mr. Etter is found to have standing under the Code then he has a right to have his appeal heard, if not then there is no right to appeal to the Council.

If Etter had some substantive basis for the appeal and he met the criteria of standing as established in the case law then I would recommend that the Council hear the matter; since standing is not conferred the Council may, if they so choose, legitimately deny the appeal.

COUNCIL, MKA: We can either deny, administratively through Marty Currie, the appeal based on lack of standing or, if Council desires, schedule the matter at the next Council meeting and discuss the matter on the record.

I am comfortable denying it administratively. Please let me know if you desire that it be brought to the hearing. I'd like to know by the budget meeting so we can inform Mr Etter in writing prior to the 21st meeting.

c Marty C
John S

KENNETH L. ETTER
P. O. Box 1653
Grand Junction, Colorado 81502

November 7, 1990

To The City of Grand Junction

Regarding Planning Commission Item #43-90

The land owner has petitioned the city for a revised final plan for the construction of a church and school. Be advised that the original conditional use permit was apparently granted in error. The subject land is in the critical zone for Walker Field runway 4-22. The original conditional use permit, granted in 1986, violated zoning criteria. A church is an incompatible use in an airport critical zone. The request for a revised final plan continues this incompatible use.

In additon, a major traffic corridor from the intersection of G Road and Horizon Drive, to about 2740 F-3/4 Road, may be essential for an adequate traffic system. This route would cross the center the subject tract of land. The city's approval of planning commission item #25-90 eliminates any other possible route for such a road (which would be perpendicular to Horizon Drive and connect G Road to F-3/4 Road).

Sincerely,

Ken Etter

Kenneth Lamar Etter

16 NOVEMBER 1990

MR. KENNETH L. ETTER
P. O. BOX 1653
GRAND JUNCTION, COLORADO 81502

RE: Planning Commission Item 43-90

Dear Mr. Etter,

This letter is being written to you in response to a letter received by the Grand Junction Community Development Department on November 8, 1990. Your letter has been forwarded on to the legal department for the purposes of addressing the validity of your claims.

Apparently your letter was an attempt at perfecting an appeal of the Planning Commission decision regarding the approval of a conditional use permit for a church at 27 1/2 Road and Cortland Avenue. On November 6, 1990, at public hearing, the Planning Commission unanimously approved the plan as submitted and granted the conditional use permit.

The Zoning and Development Code allows for appeals to the governing body (i.e. the City Council) of Planning Commission decisions when a person with standing perfects the appeal. Standing is a legal concept that in its most basic form requires that a person have some cognizable interest in the matter being decided. The concept of standing has been interpreted by the Courts to mean some form of injury in fact; a personal stake in the outcome of the controversy.

I, nor the planning staff, after hearing your testimony at the public hearing of this matter on November 6, 1990 are able to identify that you have a personal stake in the outcome of this planning item.

Therefore it is my recommendation to the Community Development Department, the City Council and the City Attorney that your appeal be deemed unperfected and denied at the administrative level for failing to be a person granted standing for purposes of an appeal under the Zoning and Development Code. The item will not be heard by the Council at the next regular meeting on November 21, 1990.

If I may be of assistance in answering questions that you may have please do not hesitate to call.

OFFICE OF THE CITY ATTORNEY
DAN E. WILSON, CITY ATTORNEY

by: /s/ 19 Nov. 90
John P. Shaver
Assistant City Attorney

xc: Martyn Currie, Interim Community Development Director
Dan E. Wilson, City Attorney

Bennett B Please handle MTA 12/20/91 c: Dan W

KENNETH L. ETTER
P. O. Box 1653
Grand Junction, Colorado 81502

December 17, 1991

Mr. Mark Achen, City Manager
City of Grand Junction
250 North 5th Street
Grand Junction, Colorado 81501-2668

Certified Mail No. P 991 544 746

Dear Mr. Achen:

Please find enclosed herewith, the following:

1) A copy of a building Permit application for construction on property at the Northeast corner of Cortland (F-3/4 Road) and 27-1/2 Road. The application is marked as being received by Mesa County on December 12, 1991.
2) A copy of a letter, dated December 16, 1991, from the Board of Mesa County Commissioners to me. The letter was signed by June Utter, Administrative Secretary. This matter seems to be in your area of responsibility.
3) A copy/reduction of a tax assessor's plat showing a proposed road traversing the building site.
4) Copies of certain portions of comments to the Metropolitan Planning Organization, dated October 1, 1990. The four selected segments are titled: G Road, Connector Road: G to F-3/4, F-3/4 Road (Cortland Ave.) and 27-1/2 Road.

The building is to be used as both a church and school. The proposed building site is directly in line with Walker Field runway 22, and this site is within the critical zone of that runway. I understand that city laws prohibit such.

An air craft which crashes on takeoff will likely be overloaded and filled with fuel. Perhaps an aviation catastrophe, as that which happened on January 13, 1982 can be avoided. In that incident, Air Florida Fight 90 crashed on takeoff at Washington, D. C. Fatalities were 74 of 79 people on the plane, and the expected fire was prevented by the Potomac River.

For both safety and traffic planning concerns, this project should be carefully reviewed. As you know, a joint resolution (Mesa County #90-97, and Grand Junction #46-90) authorized $20,000 of public funds for traffic studies. The city government seems to be obstructing these traffic studies while rejecting our offer to settle your claims. We have offered to donate this much needed road right-of-way to resolve your claims pursuant to the purported assessment recorded in book 1475, page 777.

Please review this matter.

Sincerely,

Ken Etter

Kenneth Lamar Etter

Copies: Mesa County,
Mr. Keith Mumby, counsel for Mr. E. Epstein

Grand Junction Community Development Department
Planning • Zoning • Code Enforcement
250 North Fifth Street
Grand Junction, Colorado 81501-2668
(303) 244-1430 FAX (303) 244-1599

January 6, 1992

Kenneth L. Etter
P.O. Box 1653
Grand Junction, CO 81502

Dear Mr. Etter:

This letter is in response to your letter dated Dec. 17, 1991 addressed to Mark Achen, City Manager. The 1st Presbyterian Church received a conditional use permit in 1986 allowing them to construct a church on their property at 27 1/2 Rd and Cortland (NE corner).

This site is indeed within the Walker Airport critical zone, but churches are allowed in the airport critical zone as a special use. The Conditional Use permit approval has satisfied the special use requirement.

The Conditional Use permit approved for the church allows only a church use and does not allow a school. Any use as a school would require separate approval through a revised final plan of the Conditional Use.

You are correct in stating that public funds have been set aside for traffic studies. As a matter of fact additional money besides the $20,000 has been set aside for a "roads needs study" to be completed by CRSS Civil Engineers, Inc. in 1992. A total budget of $55,480 has been budgeted for the Road Needs Study. Public input is a very important part of any study and we would encourage you to become involved. However, any new road alignments would not consider the Church property. The 1st Presbyterian Church has already received approval (since 1986) to build a church on the NE corner of 27 1/2 Rd and Cortland Avenue.

If you have any further questions, please contact us at your earliest convenience.

Respectfully,

Dave Thornton

Dave Thornton
Planner

cc: Mark Achen, City Manager

development summary

File # 43-90 Name First Presbyterian Church Date 11/06/90

PROJECT LOCATION: Northeast corner of 27 1/2 Road and Cortland Avenue.

PROJECT DESCRIPTION:

A request for a Revised Final Plan for the First Presbyterian Church on approximately 8.97 acres in a Residential Single Family (RSF-4) Zone.

REVIEW SUMMARY (Major Concerns)

POLICIES COMPLIANCE	YES	NO *	TECHNICAL REQUIREMENTS	SATISFIED	NOT SATISFIED *
Complies with adopted policies	X		Streets/Rights Of Way	X	
Complies with adopted criteria	X		Water/Sewer	X	
Meets guidelines of Comprehensive Plan	n/a		Irrigation/Drainage	X	
			Landscaping/Screening	X	
			Other:		

* See explanation below

A Conditional Use Permit was granted for the church use at this location in 1986. Because the recent plan differed significantly from the plan that was originally approved, a revised final plan process was required. This property is within the Airport Critical Zone. Currently the Zoning and Development Code does not allow churches in the Critical Zone; however, that was an oversight when the original Conditional Use was reviewed and approved. Staff has determined that there are no federal mandates prohibiting churches in the Critical Zone. The incompatibility is based solely on the potential noise conflict. Therefore, Staff will be initiating a a text amendment to remove churches as an incompatible use in the Critical Zone. The proposed development will be compatible with surrounding neighborhood.

STATUS & RECOMMENDATIONS:

Mr. Ken Etter formally appealed Planning Commission's decision of approval. Please see attached Letter. Therefore Council action is necessary.

Planning Commission Action

Planning Commission approved. (4-0)