## Fill

Fill was encountered in two borings in the eastern part of the site. The fill consisted of clayey sand with gravel, cobbles and brick debris. The fill was very loose to very dense based on field penetration resistance testing. A sample of the fill exhibited compression upon wetting under an applied pressure of 1,000 psf. Another sample tested contained 19 percent silt and clay-sized particles (passing the No. 200 sieve). The fill is likely associated with the historic use of the site. Compaction test records completed during fill placement are not likely available. The fill is considered to be of suspect quality.

## Sand and Clay

Natural clayey sand and sandy clay were found in all six borings either at the existing ground surface (beneath the pavement) or beneath the fill. The sandy clay was stiff and the clayey sand was very loose to dense. Samples of the sandy clay and clayey sand exhibited compression or low measured swell upon wetting under an applied pressure of 1,000 psf. Three samples of the clayey sand tested contained 7 to 13 percent silt and clay-size particles (passing the No. 200 sieve) and a sample of the sandy clay tested contained 68 percent silt and clay.

## Bedrock

Claystone and/or shale bedrock was encountered in all six borings beneath the surficial soils and extended to the maximum depths explored. The claystone was medium hard to very hard and the shale was very hard based on field penetration resistance tests. Samples of the claystone tested exhibited low to moderate measured swell upon wetting under an applied pressure of 1,000 psf. Two samples of claystone had unconfined compressive strengths of 18,200 and 11,900 psf. Samples of the shale were not subjected to laboratory testing due to the limited sample recovery associated with the hardness of the material.

## Ground Water

Ground water was encountered in three of our borings at depths of 7 to 21 feet during drilling. The borings were backfilled after drilling for safety reasons, preventing subsequent measurements. We understand a ground water discharge pump was installed in the southern part of the property to help lower ground water levels for an adjacent building to the south. The pump is likely drawing down ground water levels at the site.

## Seismicity

This area, like most of Colorado, is subject to a degree of seismic activity. Based upon subsurface conditions encountered during our investigation and our experience in the area, we believe the property classifies as Site Class C according to the 2009 International Building Code (IBC).

# SITE PREPARATION AND UTILITIES

Prior to construction of the proposed building, all existing structures including foundation elements, concrete flatwork, utility lines, and pavements should be removed. Existing fill is present at the site. There is a high level of risk associated with constructing foundations, slabs, and pavements on the existing fill. We recommend completely removing the existing fill beneath foundations and slabs unless deep foundations and a structural floor are used.

Where existing fill is removed, the excavation should be backfilled using imported granular fill with a maximum particle size of 2 inches. The import material should have a Liquid Limit of less than 30 percent, a Plasticity Index of less than 12 percent, and contain less than 35 percent clay and silt-size particles (passing the No. 200 sieve). A CDOT Class 5 or Class 6 base course is suitable as are many pit-run sand and gravel soils. A sample of any proposed imported fill material should be submitted to our laboratory for testing, prior to its use at the site.

Granular fill material should be moisture conditioned to within 2 percent of optimum moisture content and compacted in thin lifts to at least 98 percent of maximum standard Proctor dry density (ASTM D 698). The placement and compaction of backfill and grading fill should be observed and tested by a representative of our office during construction.

We believe the on-site materials can be excavated using conventional, heavy-duty equipment. The on-site granular soils will likely cave into unsupported, steep utility trench excavations. Based on the Occupational Safety and Health Administration (OSHA) criteria governing excavations, the existing fill and the sand and gravel materials will probably classify as Type C soils. We recommend utility trench backfill be placed in thin, loose lifts, moisture conditioned to within 2 percent of optimum moisture content, and compacted to at least 95 percent of maximum standard Proctor dry density (ASTM D 698) in structural areas and to at least 90 percent of maximum ASTM D 698 dry density in landscaped areas. Our experience suggests the on-site or imported granular soils exhibit low corrosion characteristics at natural moisture contents and under saturated conditions. The placement and compaction of the utility trench backfill should be observed and tested by a representative of our office during construction.

## FOUNDATION SYSTEMS

Considering the widely variable subsurface conditions across the site, we recommend constructing the new building on a deep foundation system such as drilled piers or micropiles. Installation of drilled piers will likely require the use of temporary casing due to the presence of granular soils and ground water, increasing cost and installation time, particularly in the eastern part of the property. The use of drilling slurry may also be required to prevent caving of the pier holes before casing installation. Locating the structure as far south and west on the property as practical should reduce problems associated with granular soils during pier installation.

Driven piles, micropiles, or helical piers could also be considered, but driven piles and helical piers may be difficult to install to the minimum recommended depths in the western part of the site due to the presence of shallow bedrock. Depending on the structural loading, micropiles may be an economical alternative.

Recommendations for drilled piers and micropiles are presented below. We recommend foundations be designed and constructed in accordance with the following criteria:

## Drilled Piers

1. Piers bottomed in the shale should be designed for a maximum allowable end pressure of 40,000 psf and an allowable skin friction of 4,000 psf for the portion of pier in shale bedrock. Skin friction should be neglected where bedrock occurs within 6 feet of the top of the pier.

2. Piers should be designed for a minimum deadload pressure of 10,000 psf based on pier cross-sectional area. If this deadload cannot be achieved through the weight of the structure, the pier length and bedrock penetration should be increased beyond the minimum values specified in the next paragraph. The bedrock should be assigned a skin friction value of 4,000 psf for uplift resistance.

3. Piers should penetrate at least 8 feet into the unweathered shale bedrock and have a minimum drilled and concreted length of at least 22 feet. The shale is very hard. A relatively large commercial size drill rig will be required to achieve minimum recommended pier lengths.

4. Piers should be reinforced their full length to resist tension in the event of swelling. We recommend the cross-sectional area of reinforcement be equal to at least 1.0 percent of the gross cross-sectional area of the pier. Grade 60 (420 Mpa) reinforcing bars (or equivalent) should be used. Reinforcement should extend into grade beams and foundation walls.

5. There should be a 6-inch (or thicker) continuous void beneath all grade beams and foundation walls, between piers, to concentrate the deadload of the structure on the piers.

6. Piers should have a center-to-center spacing of at least three pier diameters when designing for vertical loading conditions, or they should be designed as a group. Piers aligned in the direction of lateral forces should have a center-to-center spacing of at least six pier diameters. Reduction factors for closely-spaced piers are discussed in a subsequent section of the report.

7. Some movement of the drilled pier foundation is anticipated to mobilize the strength of the bedrock. We estimate this movement to be on the order of ¼ to ½-inch to mobilize skin friction. Differential movement may be equal to the total movement. Designs should consider these potential movements and accommodate them as much as practical.

8. Piers should be carefully cleaned prior to placement of concrete. Our experience indicates the presence and amount of ground water can vary significantly over relatively short vertical and horizontal distances. It is likely ground water will be encountered during drilled pier installation. Temporary casing, tremie equipment, and/or pumping may be necessary for proper cleaning, dewatering, and concrete placement. Concrete should not be placed by free fall if there is more than 3 inches of water in the bottom of the hole.

9. Concrete placed in cased pier holes should have sufficient slump to fill the pier holes and not hang on the sides of the casing or reinforcement during casing removal. We recommend a slump in the range of 5 to 7 inches if casing is used. The slump should be 4 to 6 inches for uncased piers.

10. Formation of mushrooms or enlargements at the top of piers should be avoided during pier drilling and subsequent construction operations.

11. Installation of drilled piers should be observed by a representative of our firm to identify the proper bearing strata and observe construction techniques.

## Micropiles

Micropiles are relatively small diameter (usually 3.5 to 8 inches), heavily reinforced, grouted piles. Micropiles can be constructed many ways including using open-hole methods where the hole is drilled and cuttings are flushed with air or a continuous-flight auger is used. The reinforcing is then set and grout is placed from the bottom of the hole using grout tubes. Other methods include grouting continuously with a thin grout that is pumped through hollow reinforcing steel with a sacrificial drill bit attached to the end. Considering the subsurface conditions encountered and the presence of ground water, continuous grouting will probably be required to install micropiles.

The installation methods for micropiles greatly affect the bond stress between the micropile and the surrounding soils. Design capacities must be verified in the field through full-scale load testing. Micropile construction is generally performed on a design/build basis. The micropile contractor and their engineer typically design the piles for minimum length depending on anticipated loading. We recommend micropiles penetrate a minimum of 8 feet into the relatively unweathered shale bedrock and have a minimum length of at least 22 feet. Additional guideline design criteria are provided below. We can provide contact information for some local micropile contractors if you would like.

1. Commonly available micropile systems have working capacities in the range of 20 to 100 kips. Higher overall capacities can be achieved by grouping and capping piles.

2. Micropiles should be designed and installed in accordance with Case I, Type A requirements as specified in USDOT publication FHWA-SA-97-070.

3. Micropiles should penetrate at least 8 feet into the relatively un-weathered bedrock and have a minimum length of at least 22 feet.

4. Micropiles should be reinforced their full length.

5. There should be a 6-inch (or thicker) continuous void beneath all grade beams and foundation wall, between piles.

6. Drilling methods should be determined by the contractor. Dry rotary or air flush methods are preferred to water flush.

7. Micropiles should have a minimum diameter of 4 inches. Larger sizes may be used.

8. Preliminary design should be based on a grout/ground interface bond strength of 12,000 psf in the shale bedrock and a service load factor of 2.5 (i.e. 12,000/2.5 = 4,800 psf allowable). The contractor must verify this strength is appropriate through full-scale load testing in the field.

9. Micropiles should be spaced 3 feet or more apart to avoid group efficiency effects.

10. Installation of micropiles should be observed by a representative of our firm to confirm depth and penetration into bedrock.

## Laterally Loaded Piers or Piles

Several methods are available to analyze laterally loaded piers or piles. With a pier or pile length to diameter ratio of seven or greater, we believe the method of analysis developed by Matlock and Reese is most appropriate. The method is an iterative procedure using applied lateral load, movement, vertical load, and pier diameter to develop deflection and moment versus depth curves. The computer program LPILE can be used to calculate deflections for the various pier diameters and loading conditions anticipated by the structural engineer. Suggested criteria for LPILE analysis are presented in the Table A below.

TABLE A

SOIL INPUT DATA FOR LPILE

| | Sand above Water | Sand below Water | Bedrock |
|---|---|---|---|
| Density (pci) | 0.07 | 0.03 | 0.07 |
| Cohesion, c (psi) | 0 | 0 | 45 |
| Friction Angle, φ Degrees | 32 | 32 | 0 |
| $\varepsilon_{50}$ (in/in) | - | - | 0.004 |
| $k_s$ (pci) | 90 | 60 | 2,000 |

Other procedures require input of a horizontal modulus of subgrade reaction ($K_h$). For purpose of design, we believe the soil types can be assigned the following values:

Sand above water $K_h = (21 \times z)/d$ (tons/ft$^3$)
Sand below water $K_h = (14 \times z)/d$ (tons/ft$^3$)
Bedrock $K_h = 400/d$ (tons/ft$^3$)

Where, z = depth (ft) and
d = pier or pile diameter (ft)

## Closely Spaced Pier or Pile Reduction Factors

For axial loading, a minimum spacing of three pier or pile diameters is recommended. At one diameter (touching) the skin friction load reduction factor for both piers or piles would be 0.5. End bearing values need not be reduced provided the base of the piers are at similar elevations. Interpolation can be used between one and three diameters.

Piers or piles in-line with the direction of lateral loads should have a minimum spacing of six diameters (center-to-center) based upon the larger pier or pile for use of full capacity. If a closer spacing is required, the modulus of subgrade reaction for initial and trailing elements should be reduced. At a spacing of three diameters, the effective modulus of subgrade reaction of the first pier or pile can be estimated by multiplying the given modulus by 0.6; for trailing elements in a line at a three-diameter spacing, the factor is 0.4. Linear interpolation can be used for spacing between three and six diameters.

Reductions to the modulus of subgrade reaction can be accomplished in LPILE by inputting the appropriate modification factors for the p-y curves. Reducing the modulus of subgrade reaction in trailing piers or piles will result in greater computed deflections on these elements. In practice, the grade beam can force deflections of all piers or piles to be equal. Load-deflection graphs can be generated for each pier or pile by using the appropriate p-multiplier values. The sum of the pier or pile lateral load resistances at selected deflections can be used to develop a total lateral load versus deflection graph for the system.

For lateral loads perpendicular to the line of piers or piles, a minimum spacing of three diameters can be used with no capacity reduction. At one diameter (touching) the piers can be analyzed as one unit. Interpolation can be used for intermediate conditions.

## BELOW-GRADE CONSTRUCTION

We understand no habitable below-grade areas such as a basement is planned. For areas where floors are at or above exterior grades, a foundation drain is not required.

Foundation wall backfill should be moisture conditioned to within 2 percent of optimum and compacted to at least 95 percent of maximum standard Proctor dry density (ASTM D 698). Placement and compaction of foundation wall backfill should be observed and tested by a representative of our firm during construction.

## SLABS-ON-GRADE

We expect concrete slabs-on-grade are desired for all finished floor areas. Existing fill may be present beneath slabs that are near existing grades, particularly in the eastern portion of the site. The fill is judged not suitable for support of slabs-on-grade.

The existing fill should be completely removed below slabs to expose natural sands and replaced with densely compacted granular fill. Slab subgrade fill should consist of imported granular material moisture conditioned to within 2 percent of optimum and compacted to at least 95 percent of maximum standard Proctor dry density (ASTM D 698). Imported granular fill should have a maximum particle size of 1-1/2-inch, 30 percent or less passing the No. 200 sieve, a Liquid Limit of 30 percent or less, and a Plasticity Index of 15 percent or less. A Class 5 or 6 base course will be acceptable. We recommend the building be located as far south and west on the site as possible to reduce the impact of existing fill on construction of slab-on-grade floors.

Some heave of floor slabs is likely if they are constructed near the expansive claystone. We estimate total heave of up to 2 inches is possible depending on the degree of separation between the claystone surface and slab elevation. We recommend slab-on-grade floors be separated from exterior walls and interior bearing members with joints that allow free vertical movement of slabs. Slip-joints in slab-bearing partitions should allow for at least 2 inches of free vertical movement. If the "float" is provided at the tops of partitions, the connection between interior, slab-supported partitions and exterior, foundation-supported walls should be detailed to allow differential movement. These architectural connections are critical to help reduce cosmetic damage when foundations and floor slabs move relative to each other, as happens when slabs heave. We have seen instances where these architectural connections were not designed and constructed properly and resulted in moderate cosmetic damage, even though the movement experienced was well within the anticipated range. The architect should pay special attention to these issues and detail the connections accordingly.

Frequent control joints should be provided in the slabs to reduce the effects of curling and to help control shrinkage cracking. Where underslab plumbing is necessary, service lines should be pressure tested for leaks during construction. Utility lines that penetrate the slabs should be separated and isolated from the slabs with joints to allow for free vertical movement.

The 2009 International Building Code (IBC) requires a vapor retarder be placed between base course or the subgrade soils and the concrete slab-on-grade floor, unless the designer of the floor (structural engineer) waives this requirement. The merits of installation of a vapor retarder below floor slabs depend on the sesitivity of floor coverings and building use to moisture. A properly installed vapor retarder (10 mil minimum) is more beneficial below concrete slab-on-grade floors where floor coverings, painted floor surfaces, or products stored on the floor will be sensitive to moisture. The vapor retarder is most effective when concrete is placed directly on top of it, rather than placing a sand or gravel leveling course between the vapor retarder and the floor slab. The placement of concrete on the vapor retarder may increase the risk of shrinkage cracking and curling. Use of concrete with reduced shrinkage characteristics including minimized water content, maximized coarse aggregate content, and reasonably low slump will reduce the risk of shrinkage cracking and curling. Considerations and recommendations for the installation of vapor retarders below concrete slabs are outline in Section 3.2.3 of the 2006 report of the American Concrete Institute (ACI) Committee 302, "Guide for Concrete Floor and Slab Construction (ACI 302.R-96)".

All parties must realize that even small movements of the floor slabs (less than 1-inch) can damage comparatively brittle floor treatments, such as ceramic tile. If some movement of the slabs is not acceptable, structurally supported floors with an air space between the floor and the subgrade soils are recommended. The air space required by building codes depends on the materials used to construct the floor. The structural floor is supported by the foundation system. There are design and construction issues associated with structural floors, such as ventilation and increased lateral loads, that must be considered.

## PAVEMENTS

Our exploratory borings indicate the subgrade soils beneath the proposed pavements include existing fill as well as natural sandy clay and clayey sand. The existing fill could cause higher than normal maintenance issues for pavements. Ideally, the fill should be completely removed, however, this will likely be cost prohibitive. Removing at least 2 feet of the existing fill beneath pavements and replacing it with properly moisture conditioned and compacted fill will help improve pavement performance. The existing fill may be suitable for reuse beneath pavements if it is free of debris. If it is uneconomical to remove and replace 2 feet of fill in the pavement areas, we recommend proofrolling the subgrade with a pneumatic wheeled vehicle that is loaded to at lest 18 kips per axle. Areas that are soft or exhibit significant deflection should be over-excavated or stabilized.

Considering the highly variable subsurface conditions encountered, we considered a Hveem-stabilometer ("R") value of 10 for the subgrade soils. We assigned Daily Traffic Numbers (DTN) of 2 and 5 for the parking areas and drive lanes, respectively. These values equate to equivalent 20-year, 18-kip single-axle load (ESAL) values of 14,600 and 36,500, respectively. Calculations indicate the parking areas can be paved with 5 inches of asphalt concrete or 3 inches of asphalt concrete over 6 inches of base course. Drive lanes can be paved with 6 inches of asphalt or 4 inches of asphalt over 6 inches of base course.

We recommend a plain concrete pad be provided at trash collection areas and loading docks. The pad should be at least 6 inches thick and long enough to support the entire length of the trash truck when emptying the trash dumpster. Joints between plain concrete and asphalt concrete pavements should be sealed with a flexible compound.

Our design considers pavement construction will be completed in accordance with Colorado Department of Transportation (CDOT) specifications. The specifications contain requirements for the pavement materials (asphalt concrete, base course, and plain concrete) as well as the construction practices used (compaction, materials sampling, and proof-rolling). Of particular importance are those recommendations directed toward subgrade and base course compaction and proof-rolling. During proof-rolling, particular attention should be directed toward the areas of confined backfill compaction. Areas that pump or deflect significantly should be stabilized prior to pavement construction. A representative of our office should be present at the site during construction of pavements.

## CONCRETE

Concrete in contact with soils can be subject to sulfate attack. We measured soluble sulfate concentrations from two samples from this site. The measured concentrations were 0.3 and 2.8 percent. Water-soluble sulfate concentrations in soil greater than 2 percent indicate potential for Class 3 exposure to sulfates, according to ACI 201.2R-01 as published in the 2008 ACI Manual of Concrete Practice. The American Concrete Institute (ACI) recommends using a blend of Type V cement and fly ash that meets the performance requirements (ASTM C 1012) of ACI 201,with a maximum water-to-cementitious material ratio of 0.40 and air entrainment of 5 to 7 percent for concrete with Class 3 exposure to sulfates. ACI also indicates concrete with Class 3 exposure should have a minimum compressive strength of 4,500 psi.

We understand Type V cement may not be readily available. We believe that concrete made with cement that meets ASTM C 150 Type II requirements, 20 percent fly ash, and a maximum water-to-cementitious material ratio of 0.40 can be used to provide similar resistance. The fly ash should meet ASTM C 618 Class F requirements. The fly ash content can be reduced to 15 percent for placement in cold weather months provided a water-to-cementitious material ratio of 0.40 or less is maintained. We believe this approach should be used as a minimum at this project. The more stringent measures outlined in the previous paragraph will better control risk of sulfate attack and are more in alignment with written industry standards.

## SURFACE DRAINAGE / IRRIGATION

Performance of the foundations, floor slabs, concrete flatwork, and pavements is influenced, to a large degree, by the moisture conditions existing within the near-surface soils. Overall surface drainage patterns should be planned to provide for the rapid removal of storm runoff. Water should not be allowed to pond adjacent to building foundations, over pavements, or at the crest of permanent slopes. We recommend the following precautions be observed during construction and maintained at all times after the building is completed.

1. Foundation wall backfill should be graded to provide for the rapid removal of runoff. We suggest a slope equivalent to at least 6 inches in the first 10 feet. In paved areas, the slope may be reduced to at least 2 inches in the first 10 feet.

2. Exterior foundation wall backfill should be moisture conditioned to within 2 percent of optimum and compacted to at least 90 percent of maximum standard Proctor dry density (ASTM D 698) in landscape areas and 95 percent in structural areas.

3. Roof downspouts and drains should discharge well beyond the limits of all backfill. Downspout extensions and splash blocks should be provided.

4. Landscaping concepts should concentrate on use of native plants that require little or no supplemental irrigation after the establishment period.

## CONSTRUCTION OBSERVATIONS

We recommend that CTL|Thompson, Inc. provide observation and testing services during construction to allow us the opportunity to verify whether soil conditions are consistent with those found during this investigation. If others perform these observations, they must accept responsibility to judge whether the recommendations in this report remain appropriate.

## GEOTECHNICAL RISK

The concept of risk is an important aspect with any geotechnical evaluation primarily because the methods used to develop geotechnical recommendations do not comprise an exact science. We never have complete knowledge of subsurface conditions. Our analysis must be tempered with engineering judgment and experience. Therefore, the recommendations presented in any geotechnical evaluation should not be considered risk-free. Our recommendations represent our judgment of those measures that are necessary to increase the chances that the structure will perform satisfactorily. It is critical that all recommendations in this report are followed during construction.

## LIMITATIONS

Our borings were located to obtain a reasonably accurate indication of subsurface foundation conditions. The borings are representative of conditions encountered at the exact boring location only. Variations in subsurface conditions not indicated by the borings are possible. A representative of our firm should provide construction observation and materials testing services during construction.

We believe this investigation was conducted with that level of skill and care normally used by geotechnical engineers practicing in this area at this time. No warranty, express or implied, is made. If we can be of further service in discussing the contents of this report, or in the proposed construction from a geotechnical point of view, please call.

CTL | THOMPSON, INC.

COLORADO REGISTERED PROFESSIONAL ENGINEER
MICHAEL NEAL LEMONS
33394
6/9/11

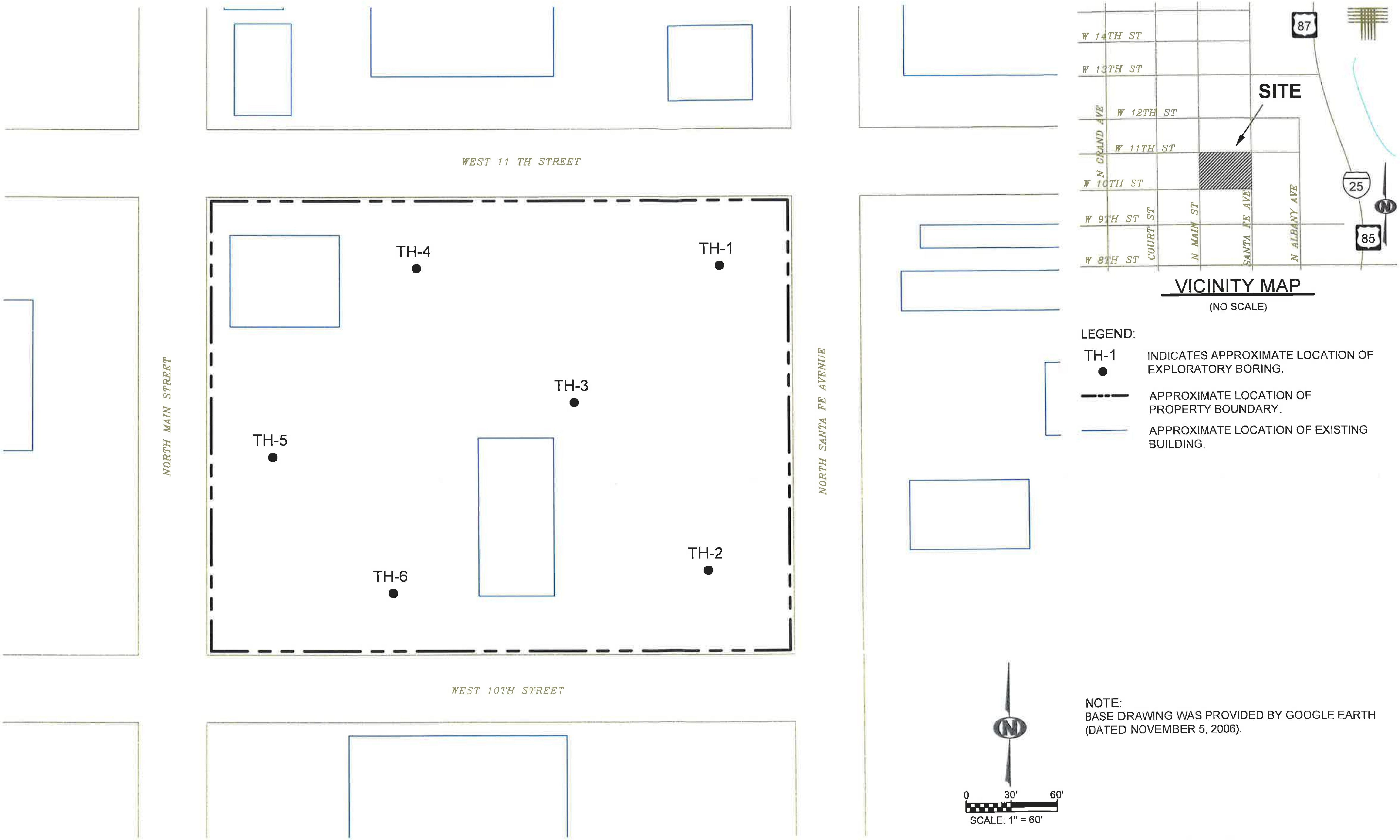
Michael N. Lemons, P.E.
Associate Engineer

Reviewed by:

Richard A. Phillips, P.E.
Senior Principal Engineer

MNL:RAP:eas

(4 copies sent)




**Location of Exploratory Borings**

FIG. 1

PUEBLO COUNTY
PUEBLO COUNTY EMERGENCY SERVICES CENTER
CTL|T PROJECT NO. SC02821-125
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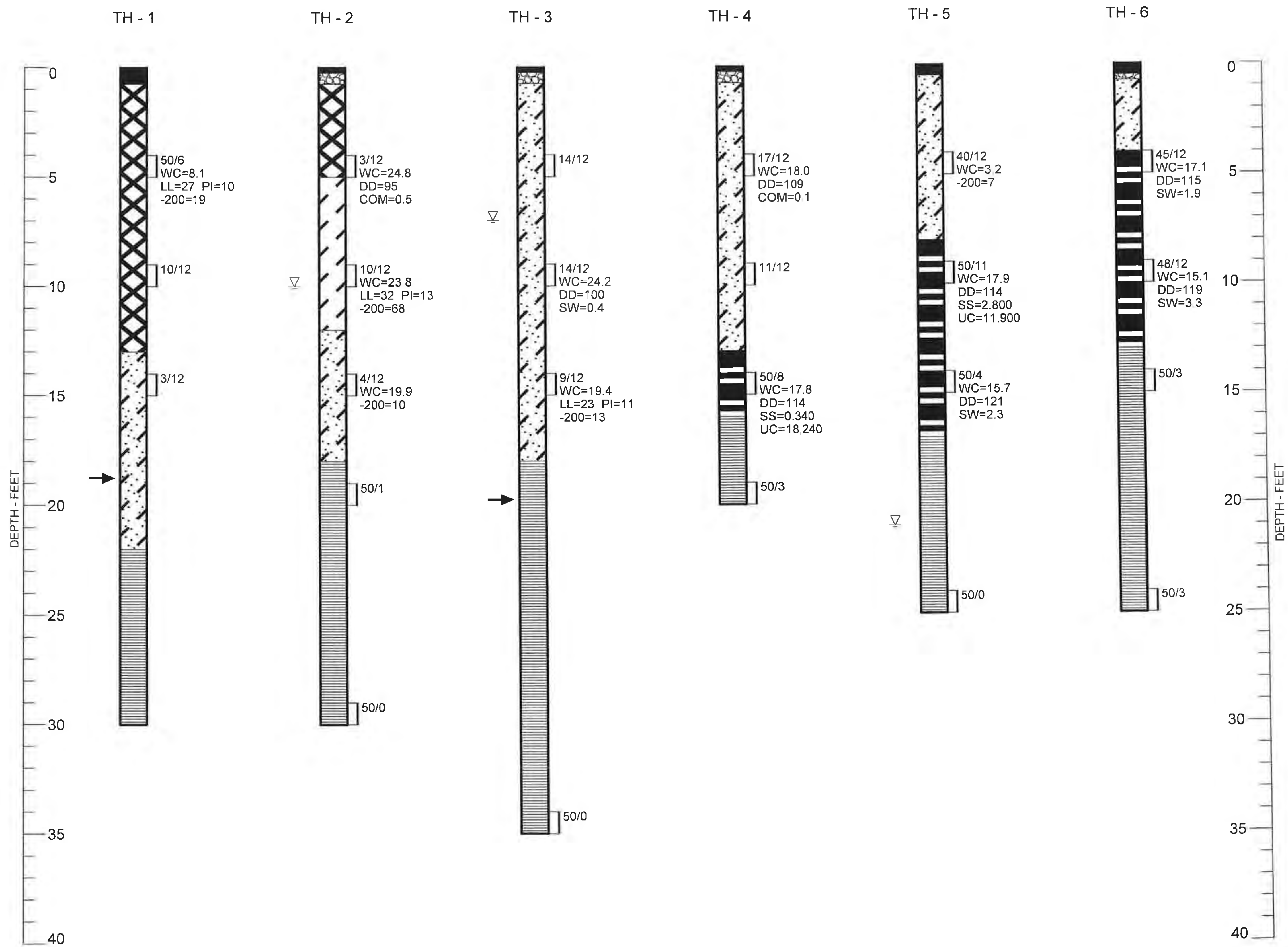


**LEGEND:**

ASPHALT PAVEMENT

BASE COURSE.

FILL, SAND, CLAYEY WITH GRAVELS AND COBBLES, BRICK DEBRIS, VERY LOOSE TO VERY DENSE, SLIGHTLY MOIST TO MOIST, BROWN.

SAND, CLAYEY, GRAVELLY, VERY LOOSE TO DENSE, SLIGHTLY MOIST TO WET, BROWN (SC)

CLAY, SANDY, STIFF, SLIGHTLY MOIST TO WET, BROWN (CL).

CLAYSTONE, MEDIUM HARD TO VERY HARD, SLIGHTLY MOIST TO MOIST, BROWN TO GRAY BROWN.

SHALE, VERY HARD, SLIGHTLY MOIST, GRAY TO DARK GRAY.

DRIVE SAMPLE. THE SYMBOL 50/6 INDICATES 50 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2.5-INCH O.D. SAMPLER 6 INCHES.

GROUND WATER LEVEL MEASURED AT TIME OF DRILLING.

INDICATES DEPTH WHERE THE TEST HOLE CAVED DURING DRILLING.

**NOTES:**

1. THE BORINGS WERE DRILLED APRIL 29, 2011 USING A 4-INCH DIAMETER, CONTINUOUS-FLIGHT AUGER AND A CME-55, TRUCK-MOUNTED DRILL RIG.
2. THESE LOGS ARE SUBJECT TO THE EXPLANATIONS, LIMITATIONS, AND CONCLUSIONS AS CONTAINED IN THIS REPORT.
3. WC - INDICATES MOISTURE CONTENT. (%)
   DD - INDICATES DRY DENSITY. (PCF)
   SW - INDICATES SWELL WHEN WETTED UNDER 1 KSF LOAD. (%)
   COM - INDICATES COMPRESSION WHEN WETTED UNDER 1 KSF LOAD. (%)
   LL - INDICATES LIQUID LIMIT. (%) (NV : NO VALUE)
   PI - INDICATES PLASTICITY INDEX. (%) (NP : NON-PLASTIC)
   -200 - INDICATES PASSING NO. 200 SIEVE. (%)
   SS - INDICATES WATER-SOLUBLE SULFATE CONTENT. (%)
   UC - INDICATES UNCONFINED COMPRESSIVE STRENGTH. (PSF)

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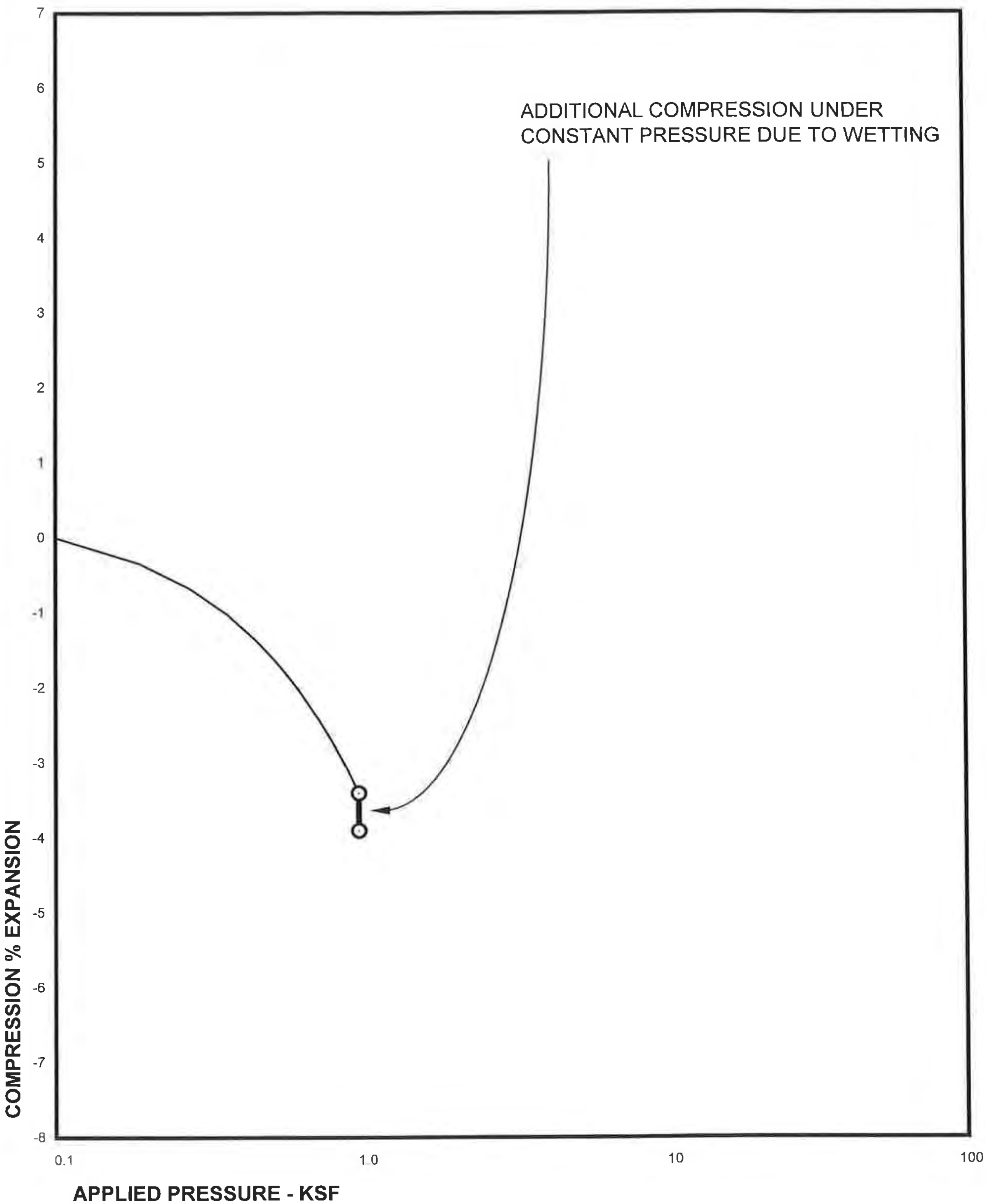
## Summary Logs of Exploratory Borings

FIG. 2




| | | | | | |
|---|---|---|---|---|---|
| Sample of | FILL, SAND, CLAYEY | | DRY UNIT WEIGHT= | 95 | PCF |
| From | TH-2 AT 4 FEET | | MOISTURE CONTENT= | 24.8 | % |

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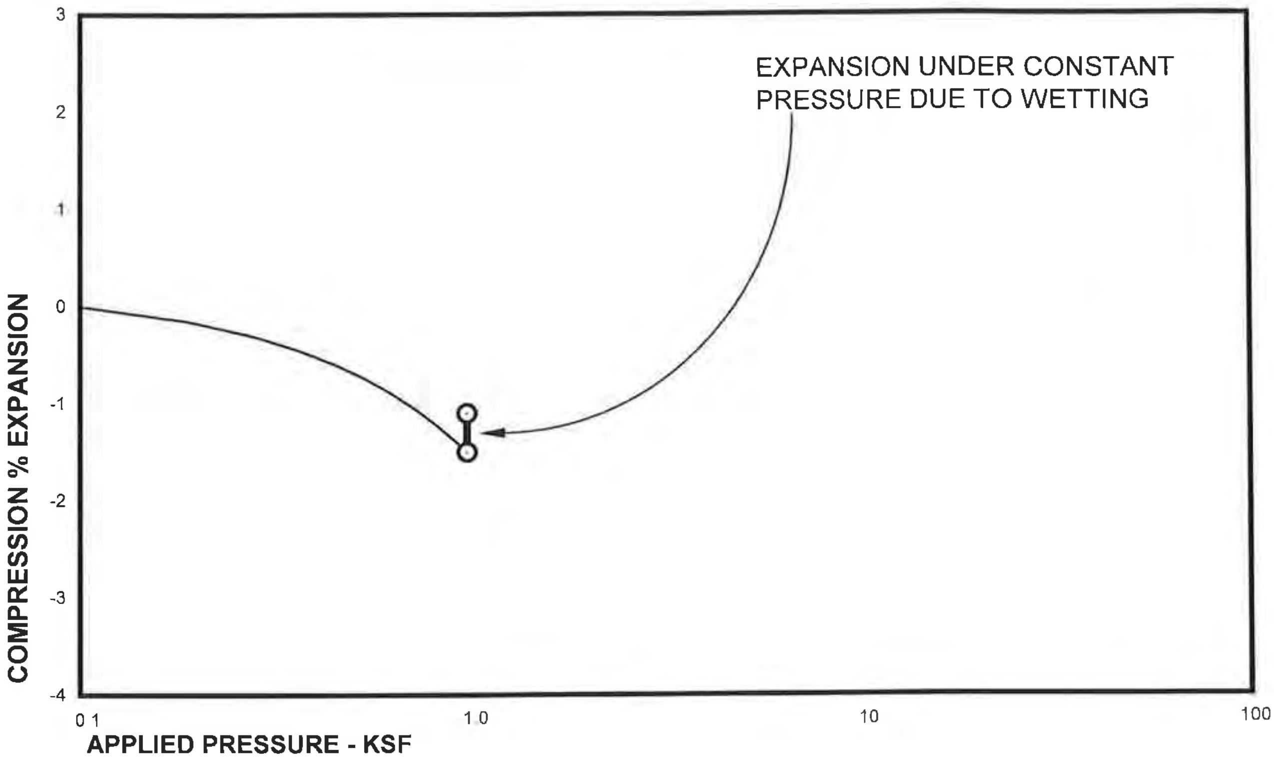
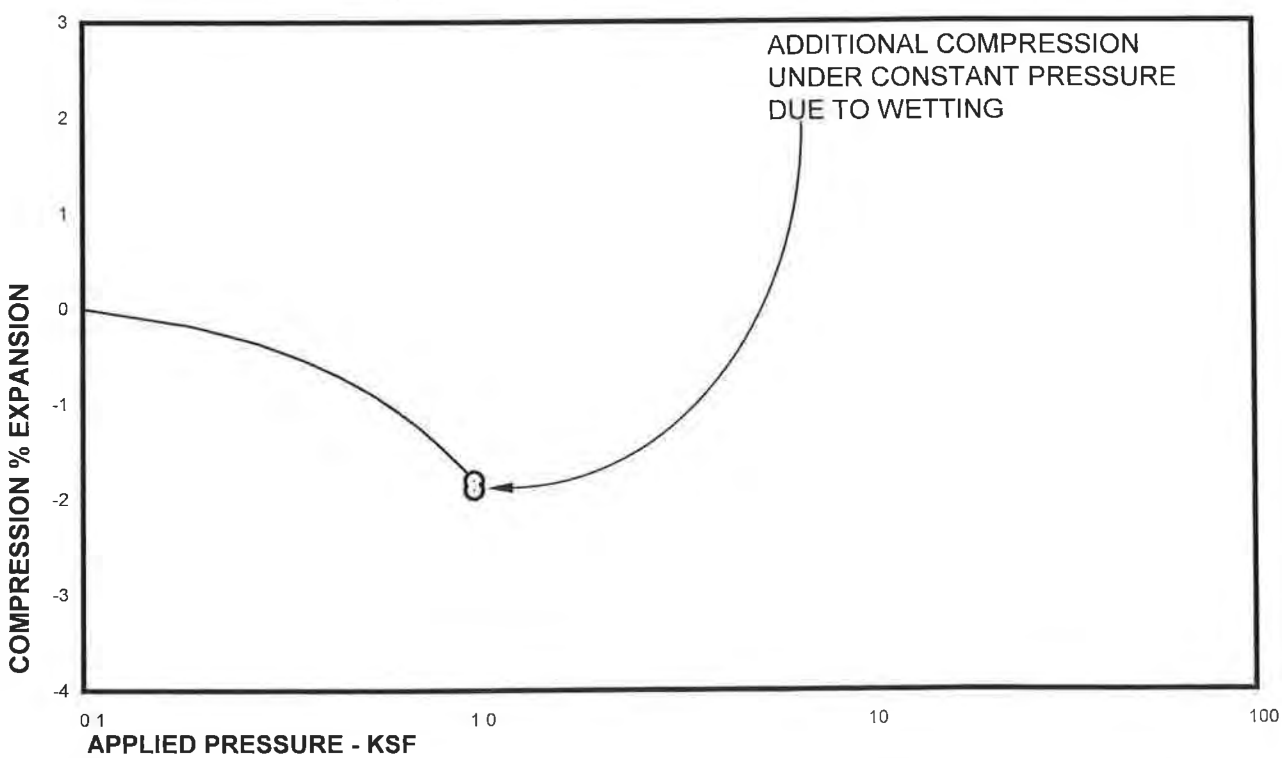
## Swell Consolidation Test Results

FIG. 3




Sample of SAND, CLAYEY (SC)
From TH-3 AT 9 FEET

DRY UNIT WEIGHT= 100 PCF
MOISTURE CONTENT= 24.2 %




Sample of SAND, CLAYEY (SC)
From TH-4 AT 4 FEET

DRY UNIT WEIGHT= 109 PCF
MOISTURE CONTENT= 18.0 %

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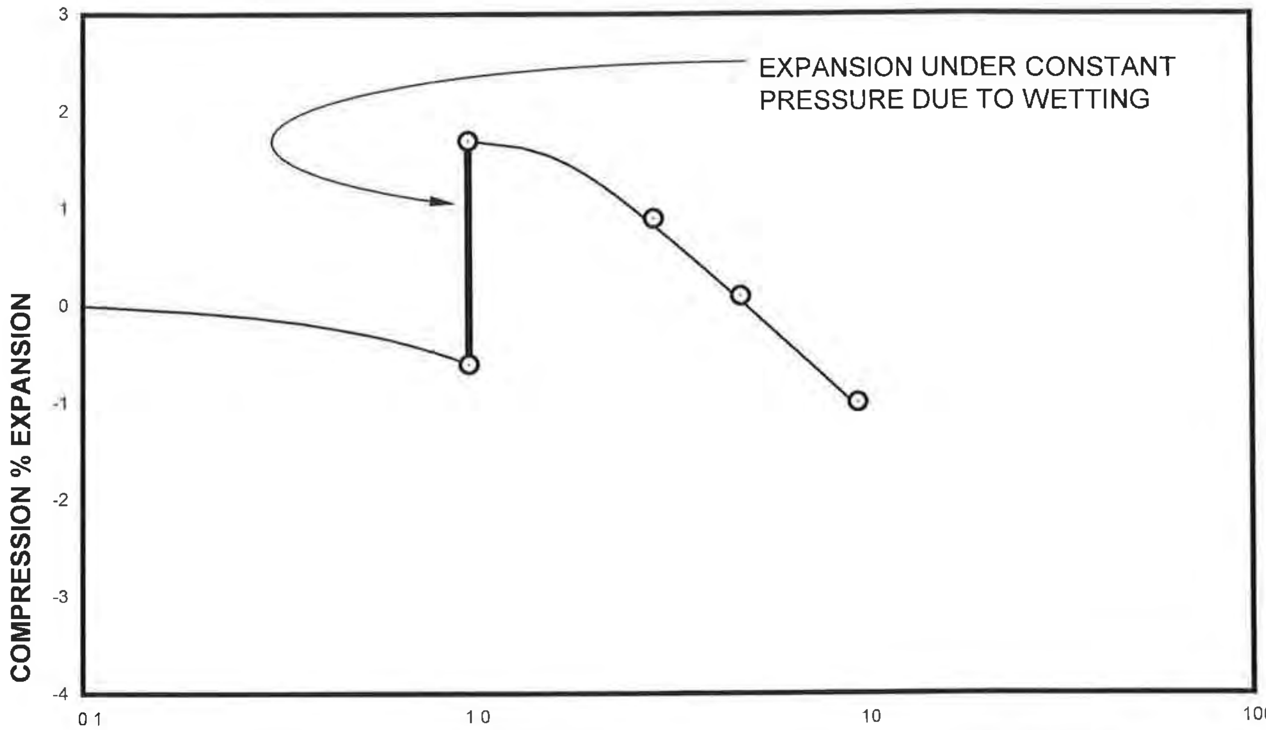
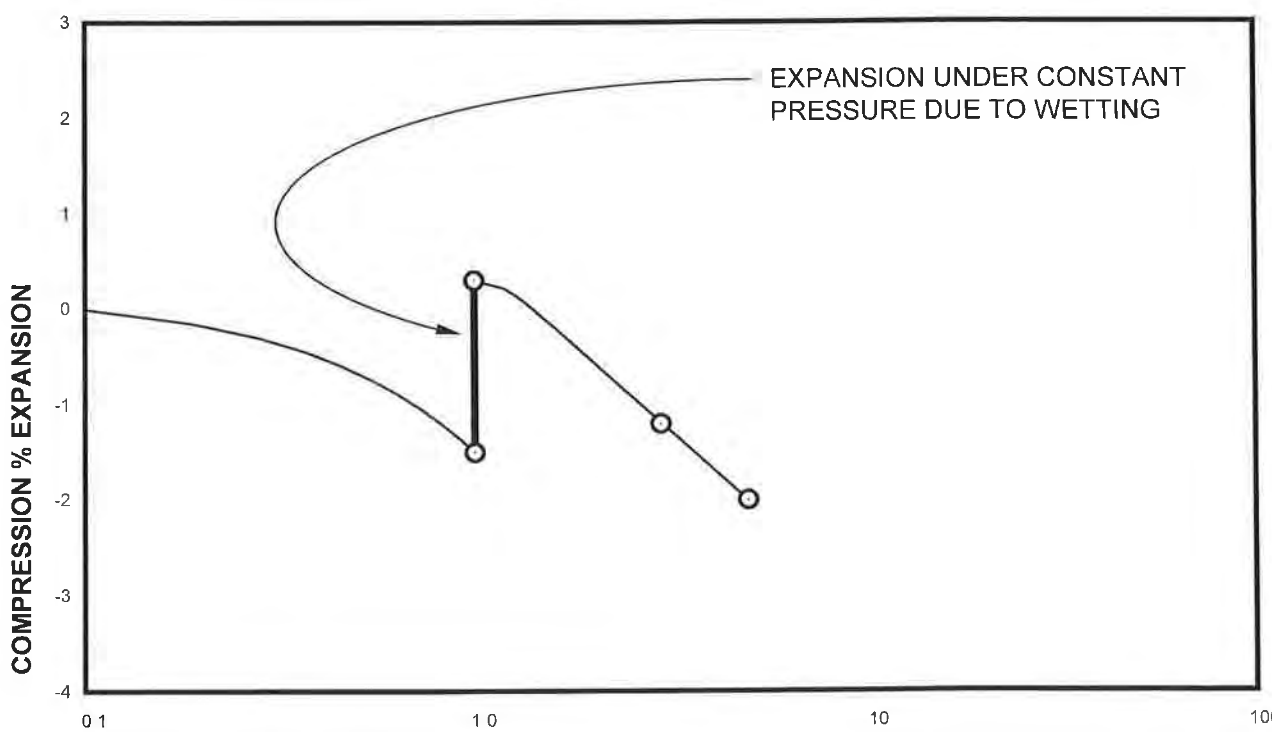
## Swell Consolidation Test Results

FIG. 4




| | | | | |
|---|---|---|---|---|
| Sample of | CLAYSTONE | DRY UNIT WEIGHT= | 121 | PCF |
| From | TH-5 AT 14 FEET | MOISTURE CONTENT= | 15.7 | % |




| | | | | |
|---|---|---|---|---|
| Sample of | CLAYSTONE | DRY UNIT WEIGHT= | 115 | PCF |
| From | TH-6 AT 4 FEET | MOISTURE CONTENT= | 17.1 | % |

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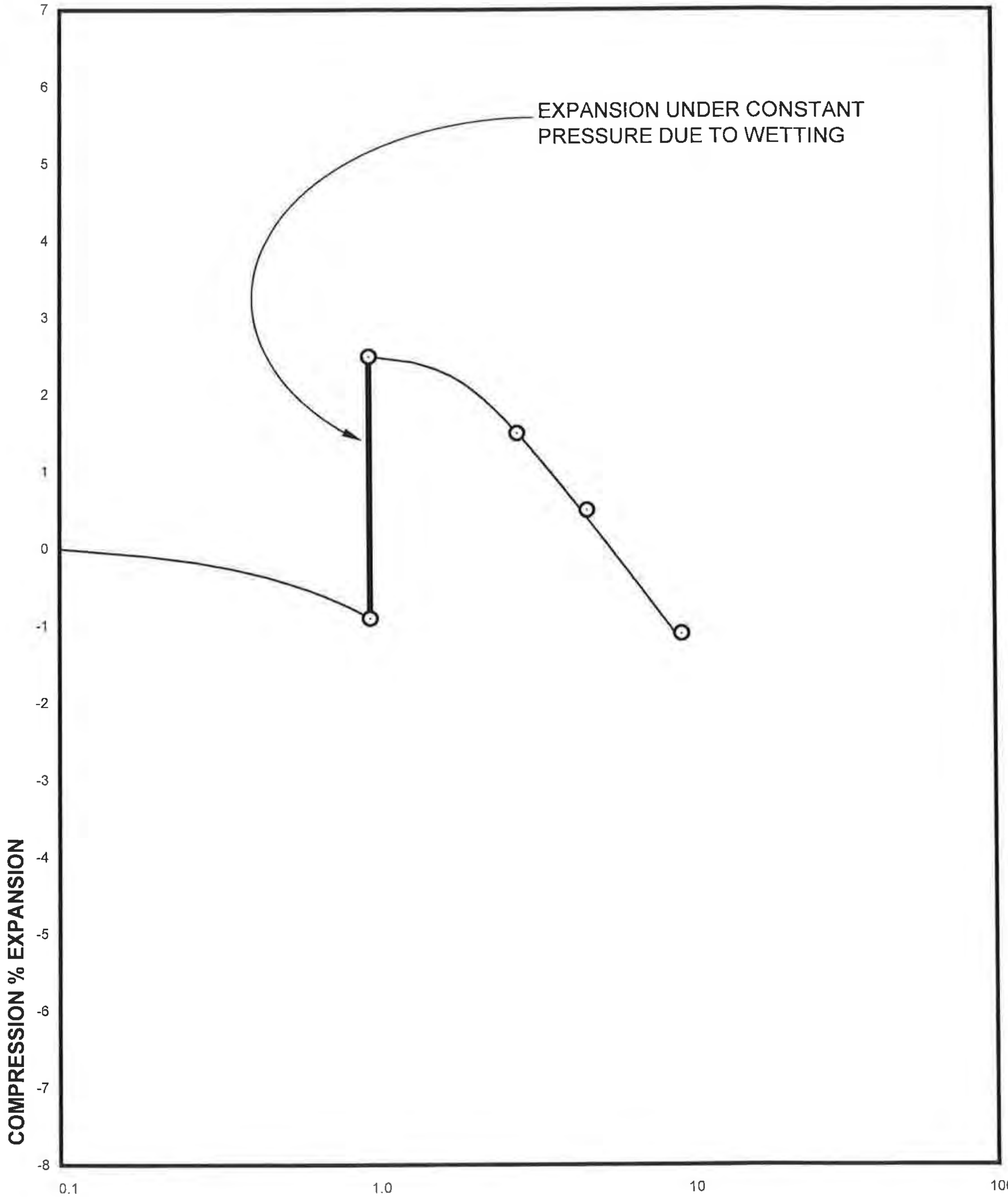
## Swell Consolidation Test Results

FIG. 5

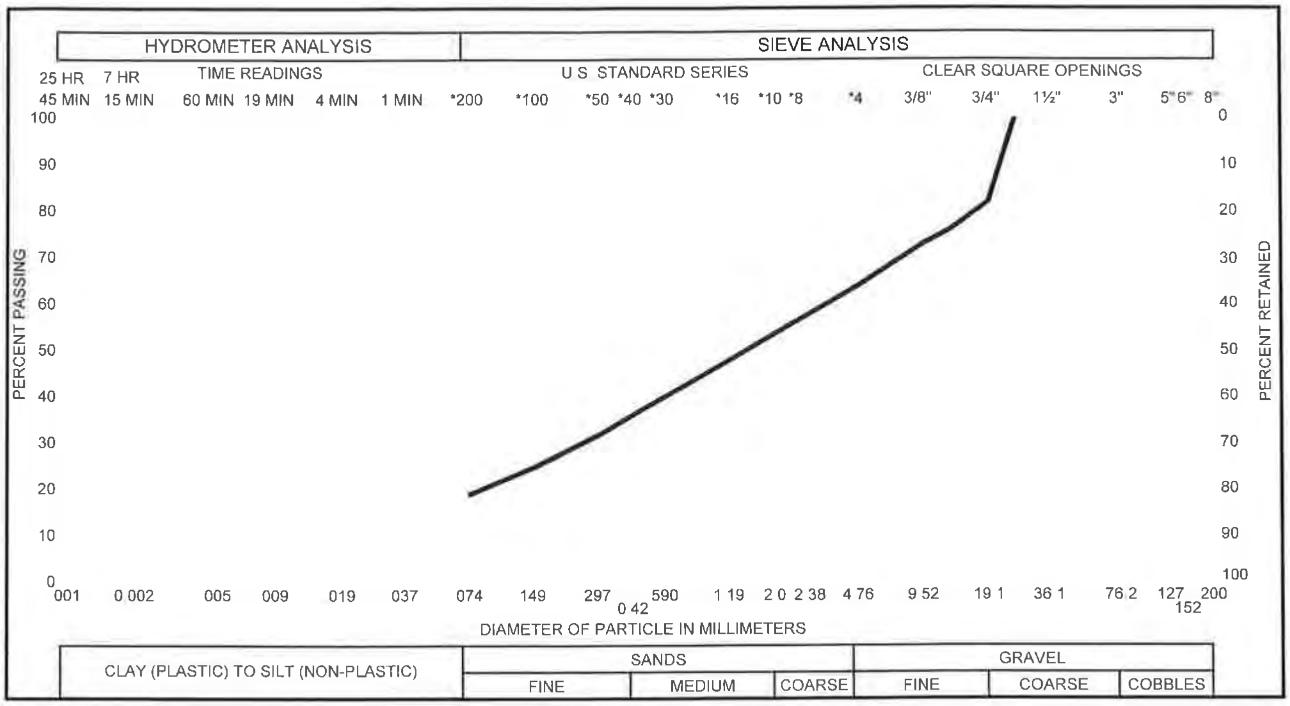



Sample of CLAYSTONE
From TH-6 AT 9 FEET

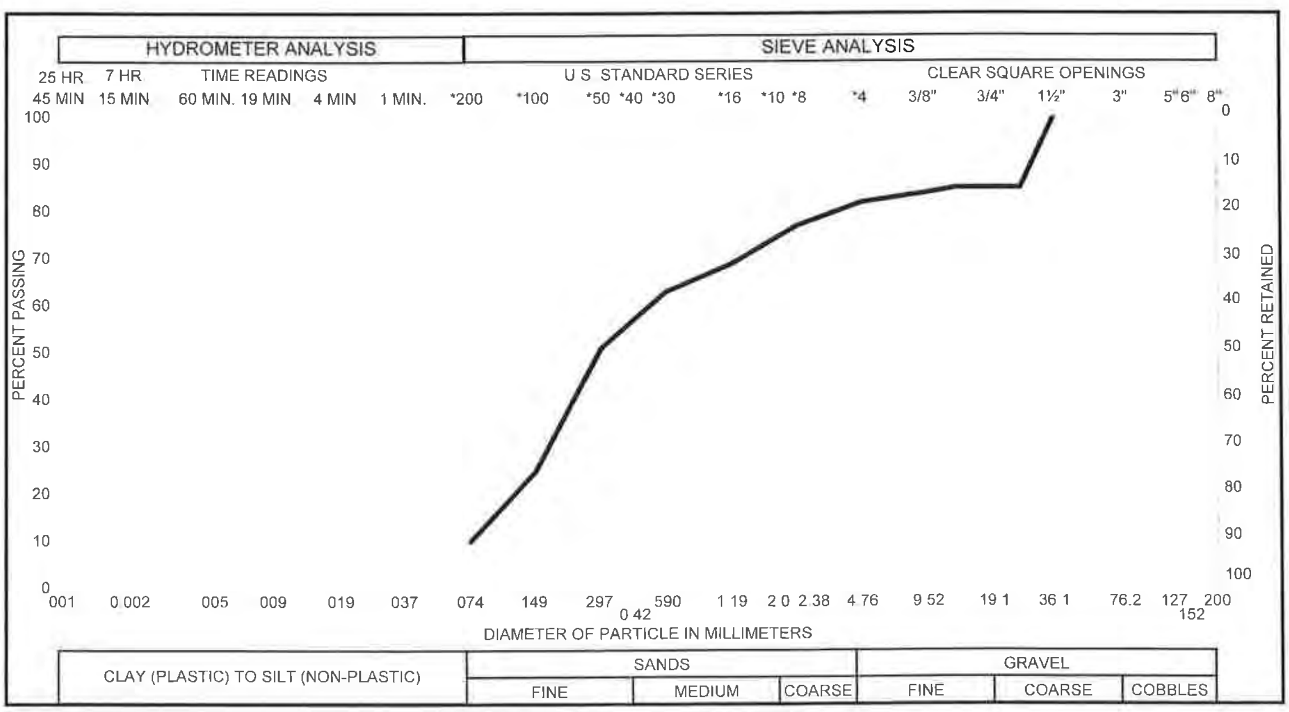
DRY UNIT WEIGHT= 119 PCF
MOISTURE CONTENT= 15.1 %

PUEBLO COUNTY
PUEBLO COUNTY EMERGENCY SERVICES CENTER
CTL|T PROJECT NO. SC02821-125
S:\SC-2800-2899\SC02821.000\125\2. Reports\SC02821-125_SWELL.xls

## Swell Consolidation Test Results




| | | | | |
|---|---|---|---|---|
| Sample of | FILL, SAND, GRAVELLY, CLAYEY | GRAVEL 36 % | SAND | 45 % |
| From | TH - 1 AT 4 FEET | SILT & CLAY 19 % | LIQUID LIMIT | 27 % |
| | | PLASTICITY INDEX | | 10 % |

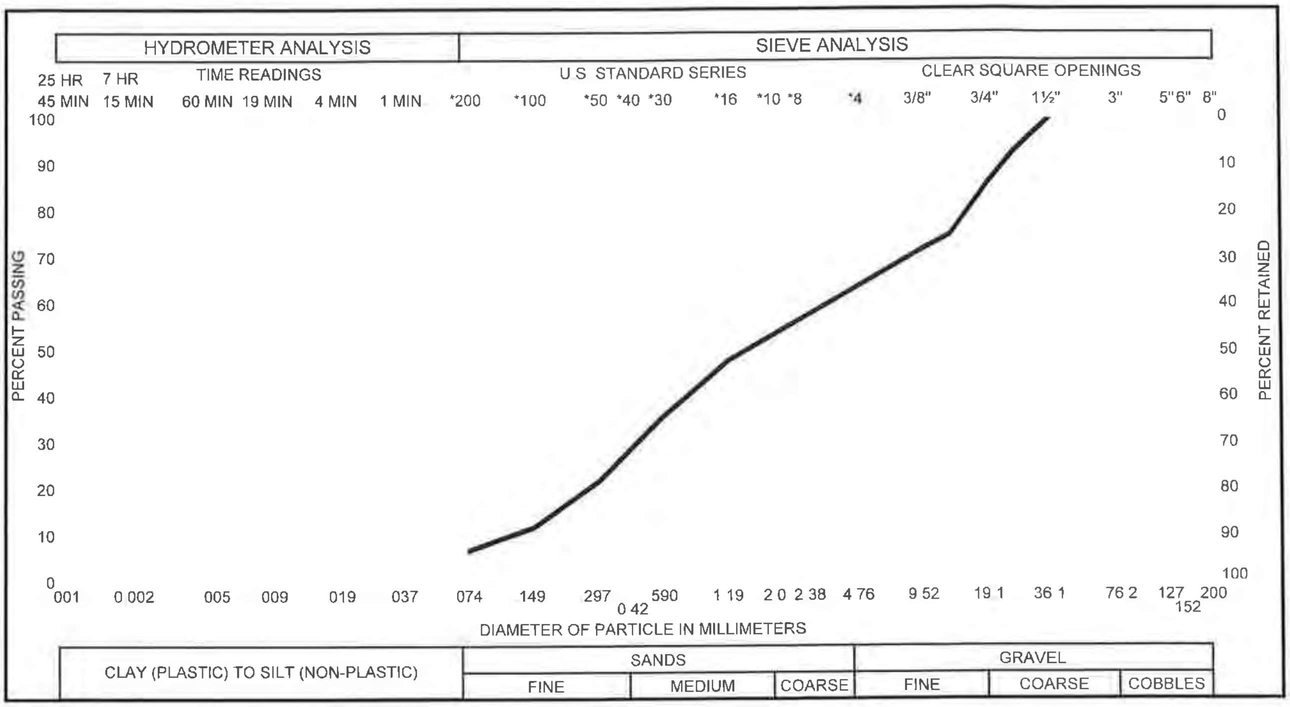



| | | | | |
|---|---|---|---|---|
| Sample of | SAND, GRAVELLY, CLAYEY (SP-SC) | GRAVEL 18 % | SAND | 72 % |
| From | TH 2 AT 14 FEET | SILT & CLAY 10 % | LIQUID LIMIT | % |
| | | PLASTICITY INDEX | | % |



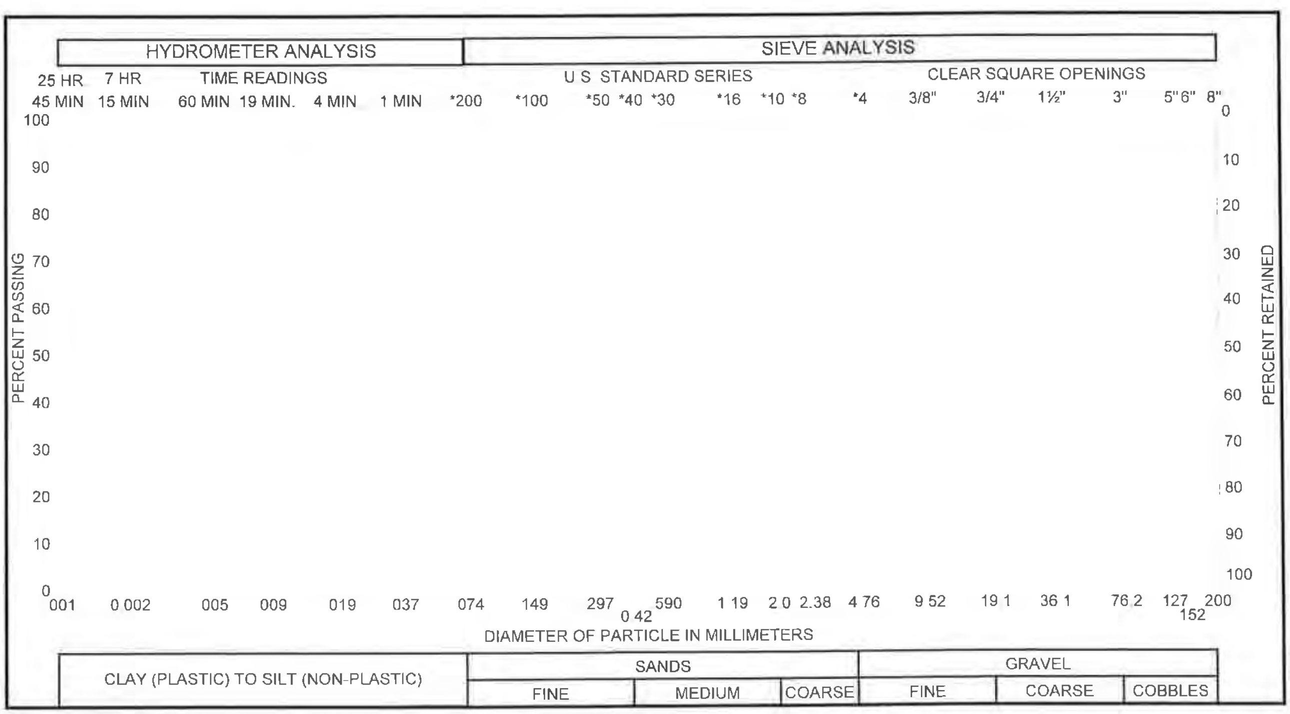
# Gradation Test Results

FIG. 7




Sample of SAND, GRAVELLY, CLAYEY (SP-SC)
From TH 5 AT 4 FEET

GRAVEL 36 % SAND 57 %
SILT & CLAY 7 % LIQUID LIMIT %
PLASTICITY INDEX %




Sample of
From

GRAVEL % SAND %
SILT & CLAY % LIQUID LIMIT %
PLASTICITY INDEX %

**PUEBLO COUNTY**
**PUEBLO COUNTY EMERGENCY SERVICES CENTER**
**PROJECT NO. SC02821-125**
**S:\SC-2800-2899\SC02821.000\125\2. Reports\SC02821-125_Gradation.xls**

# Gradation Test Results

FIG. 8

# TABLE 1

**SUMMARY OF LABORATORY TESTING**
**CTL|T PROJECT NO. SC02821-125**

| BORING | DEPTH (FEET) | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | ATTERBERG LIMITS LIQUID LIMIT (%) | ATTERBERG LIMITS PLASTICITY INDEX (%) | SWELL TEST RESULTS* SWELL (%) | SWELL TEST RESULTS* SWELL PRESSURE (PSF) | UNCONFINED COMPRESSION (PSF) | PASSING NO. 200 SIEVE (%) | SOLUBLE SULFATES (%) | DESCRIPTION |
|---|---|---|---|---|---|---|---|---|---|---|---|
| TH-1 | 4 | 8.1 | | 27 | 10 | | | | 19 | | FILL, SAND, GRAVELLY, CLAYEY |
| TH-2 | 4 | 24.8 | 95 | | | -0.5 | | | | | FILL, SAND, CLAYEY |
| TH-2 | 9 | 23.8 | | 32 | 13 | | | | 68 | | CLAY, SANDY (CL) |
| TH-2 | 14 | 19.9 | | | | | | | 10 | | SAND, GRAVELLY, CLAYEY (SP-SC) |
| TH-3 | 9 | 24.2 | 100 | | | 0.4 | | | | | SAND, CLAYEY (SC) |
| TH-3 | 14 | 19.4 | | 23 | 11 | | | | 13 | | SAND, CLAYEY (SC) |
| TH-4 | 4 | 18.0 | 109 | | | -0.1 | | | | | SAND, CLAYEY (SC) |
| TH-4 | 14 | 17.8 | 114 | | | | | 18,200 | | 0.3 | CLAYSTONE |
| TH-5 | 4 | 3.2 | | | | | | | 7 | | SAND, GRAVELLY, CLAYEY (SP-SC) |
| TH-5 | 9 | 17.9 | 114 | | | | | 11,900 | | 2.8 | CLAYSTONE |
| TH-5 | 14 | 15.7 | 121 | | | 2.3 | 8,000 | | | | CLAYSTONE |
| TH-6 | 4 | 17.1 | 115 | | | 1.9 | 4,000 | | | | CLAYSTONE |
| TH-6 | 9 | 15.1 | 119 | | | 3.3 | 9,000 | | | | CLAYSTONE |

* SWELL MEASURED WITH 1000 PSF APPLIED PRESSURE, OR ESTIMATED IN-SITU OVERBURDEN PRESSURE.
NEGATIVE VALUE INDICATES COMPRESSION.

# *EXHIBIT 3: LOMR CORRESPONDENCE*

# Federal Emergency Management Agency

Washington, D.C. 20472

CERTIFIED MAIL
RETURN RECEIPT REQUESTED

IN REPLY REFER TO:
102
Case No.: 90-08-34P
Community: City of Pueblo, Colorado
Map Panel Numbers:
FIRM: 085077 0006 C, 085077 0010 C
FBFM: 085077 0006, 085077 0010
Effective Date
of This Revision: APR 25 1991

The Honorable Michael A. Occhiato
President of the Council
City of Pueblo
P.O. Box 1427
Pueblo, Colorado 81002

Dear President Occhiato:

This is in response to a letter dated September 13, 1990, from Mr. Dennis A. Maroney, Drainage Engineer, City of Pueblo, regarding the effective Flood Insurance Study (FIS) report, Flood Insurance Rate Map (FIRM), and Flood Boundary and Floodway Map (FBFM) for the City of Pueblo, Colorado. Mr. Maroney requested that we revise the effective FIRM and FBFM to show the effects of a channel improvement and levee project along a reach of Fountain Creek from the confluence with the Arkansas River upstream to U.S. Highway 50.

All data required to process this request were submitted by you with your letter dated April 16, 1990; by Mr. Gary L. Eyster, P.E., Chief, Planning Branch, U.S. Army Corps of Engineers (COE), Albuquerque District, with his letters dated July 24 and August 7, 1990; and by Mr. Dennis A. Maroney, with letters dated between September 13, 1990, and March 29, 1991.

As part of this project, levees and a floodwall were built by the COE along the designated reaches of the eastern and western banks of Fountain Creek. The western levee is located between Interstate 25 and the Denver and Rio Grande Western Railroad from just downstream of Seventh Street to just upstream of Eighth Street, and ties into a floodwall which extends approximately 800 feet upstream to approximately 11th Street; the eastern levee extends from approximately 500 feet north of 13th Street downstream to the Missouri and Pacific Railroad (MPRR) bridge abutment. As a result of this project, flooding from the Fountain Creek East Bank Overflow located on the eastern boundary has been eliminated. Three ponded areas (designated as Zone A) have been added to the FIRM near Beech Street and Joplin Avenue, due to interior drainage behind the eastern levee. Flooding which was the result of breakout along the western boundary of Fountain Creek from approximately 14th Street to the Arkansas River also has been eliminated as a result of this project.

All flooding is contained within the limits of the Fountain Creek Flood Control Project. The floodplain boundaries have either decreased or remained the same as those shown on the effective FIRM, except for the area along the west bank in the vicinity of Chester Avenue between Fifth and Eighth Streets and for the annexed area in the vicinity of Joplin Avenue and the MPRR. All land within the Fountain Creek Flood Control Project limits is owned by the City of Pueblo, with the exception of three structures in the Chester Avenue area.

In the February 14, 1991, letter, Mr. Maroney stated that the area near Chester Avenue between Fifth and Eighth Streets falls within an existing floodplain district, S-3 Zone. It is our understanding that no additions to existing buildings or future construction of buildings are allowed within this zone. Therefore, the change in floodplain boundaries does not place additional building restrictions on any structures located within the S-3 Zone.

According to the agreement for project acceptance, the City will maintain the project. Maintenance of the project stipulates that the City of Pueblo will not allow any development or fill, or construction within the limits of the project. The City must also maintain the channel portions of the project at the elevation they were when the project was completed. These restrictions are compatible with existing floodway criteria and restrictions currently enforced by the City of Pueblo.

Because all flooding is within the project limits, no regulatory floodway is shown downstream of U.S. Highway 50. The existing floodway for Fountain Creek has been deleted from U.S. Highway 50 downstream to the corporate limits. Cross sections A through K have been eliminated from the Floodway Data Table and on Profile Panels 02P through 04P for this reach of Fountain Creek.

The land in the vicinity of Joplin Avenue and the MPRR was annexed from the unincorporated areas of Pueblo County by the City. However, a portion of this annexation west of Joplin Avenue is not shown on the effective FIRM for Pueblo County, Colorado, dated September 29, 1989. This area is now designated as a Special Flood Hazard Area (SFHA). The detailed flooding associated with this area is confined to the limits of the Fountain Creek Flood Control Project and the land is owned by the City as stated in the March 29, 1991, letter from Mr. Maroney. The remaining SFHA annexed east of Joplin Avenue is designated as Zone A approximate, based on information from the Pueblo County FIRM and is also owned by the City.

We have completed our review of the submitted data with regard to the data used to produce the effective FIRM and FBFM and have revised the FIRM and FBFM to modify the floodplain boundary delineations and floodway boundary delineations of a flood having a 1-percent probability of being equaled or exceeded in any given year (base flood) along Fountain Creek from its confluence with the Arkansas River (near Joplin Avenue) to U.S. Highway 50.

The modifications are shown on the enclosed annotated copies of FIRM Panels 0006 and 0010, FBFM Panels 0006 and 0010, Floodway Data Tables for Fountain Creek and Fountain Creek East Bank Overflow, Summary of Discharges Table, Flood Hazard Data Table and Profile Panels 02P through 04P for Fountain Creek. This Letter of Map Revision (LOMR) hereby revises these panels of the effective FIRM and FBFM, both dated September 29, 1986. Profile Panel 06P and the floodway for Fountain Creek East Bank Overflow have been deleted.

Because of current funding constraints, we must limit the number of physical map revisions. Consequently, we will not publish a revised FIRM and FBFM for the City of Pueblo to reflect modifications at this time. However, if in the future we revise and republish the FIRM and FBFM panels affected by this LOMR, we will incorporate the previously described modifications at that time.

The following table is a partial listing of former and modified 100-year flood elevations for Fountain Creek.

| Location | Existing Base Flood Elevation *(feet) | Modified Base Flood Elevation *(feet) | |
|---|---|---|---|
| Just upstream of Joplin Avenue | None | 4,644 | |
| Just downstream of Missouri Pacific Railroad | 4,656 | 4,651 | -5 |
| Just downstream of Fourth Street | 4,676 | 4,676 | |
| Just upstream of Eighth Street | 4,687 | 4,683 | -4 |
| Just downstream of U.S. Highway 50 | 4,702 | 4,702 | |

*National Geodetic Vertical Datum, rounded to the nearest whole foot.

In general, a floodway is provided to your community as a tool to regulate floodplain development. Therefore, the floodway modifications described in this letter, while acceptable to the Federal Emergency Management Agency (FEMA), must also be acceptable to your community and adopted by appropriate community action, as specified in Paragraph 60.3(d) of the National Flood Insurance Program (NFIP) regulations.

These modifications have been made pursuant to Section 206 of the Flood Disaster Protection Act of 1973 (P.L. 93-234) and are in accordance with the National Flood Insurance Act of 1968, as amended (Title XIII of the Housing and Urban Development Act of 1968, P.L. 90-448), 42 U.S.C. 4001-4128, and 44 CFR, Part 65. Public notification of modifications to the base (100-year) flood elevations (BFEs) along Fountain Creek will be given in the Pueblo Chieftain and Star Journal on or about May 10 and May 17, 1991. In addition, a Notice of Changes will be published in the Federal Register.

As required by the legislation, a community must adopt and enforce floodplain management measures to ensure continued eligibility to participate in the NFIP. Therefore, your community must enforce these regulations using, at a minimum, the BFEs, zone designations, and floodways in the Special Flood Hazard Areas shown on the FIRM and FBFM for your community, including the previously described modifications.

This response to your request is based on minimum floodplain management criteria established under the NFIP. Your community is responsible for approving all proposed floodplain developments, including this request, and for ensuring that necessary permits required by Federal or State law have been received. With knowledge of local conditions and in the interest of safety, State and community officials may set higher standards for construction, or may limit development in floodplain areas. If the State of Colorado or the City of Pueblo has adopted more restrictive or comprehensive floodplain management criteria, these criteria take precedence over the minimum NFIP requirements.

The basis of this LOMR is, in part, a channel-modification project. NFIP regulations, as cited in Section 60.3(b)(7), require that communities assure that the flood-carrying capacity within the altered or relocated portion of any watercourse is maintained. This provision is incorporated into your community's existing floodplain management regulations. Consequently, the ultimate responsibility for maintenance of the channel modification rests with your community.

The community number and suffix code listed above will be used for all flood insurance policies and renewals issued for your community on and after the effective date listed above.

The modifications described herein are effective as of the date of this letter. However, within 90 days of the second publication in the Pueblo Chieftain and Star Journal, your community may request that we reconsider this determination.. Any request for reconsideration must be based on scientific or technical data. All interested parties are hereby notified that, until the 90-day period elapses, the determination may be modified.

If you have any questions regarding the modifications described herein, please call the Chief, Natural and Technological Hazards Division, FEMA, in Denver, Colorado, at (303) 235-4830, or Mrs. Cynthia M. Croxdale of my staff in Washington, D.C., at (202) 646-3458.

Sincerely,

William R. Locke

William R. Locke
Chief, Risk Studies Division
Federal Insurance Administration

Enclosures

cc: Mr. Dennis A. Maroney
Drainage Engineer
City of Pueblo Department
of Public Works

Ms. Lisa A. De Bettignies
U.S. Army Corps of Engineers
Albuquerque District

| | |
|---|---|
| bcc: State Coordinator | |
| Regional Director | R-8-NT |
| Virginia Motoyama | R-8-NT |
| Originator | IA-RA-TO |
| Office Chron | IA-RA-RS |
| MBJ File/MCM/JCB | |

| Concurrence: | IA-RA-TO | IA-RA-RS | IA-RA-RS |
|---|---|---|---|
| | Flowers | Croxdale | Mohr |
| | | cmc 4/23/91 | |

Table 2. Summary of Discharges

| Flooding Source and Location | Drainage Area (Square Miles) | Peak Discharges (cfs) 10-Year | 50-Year | 100-Year | 500-Year |
|---|---|---|---|---|---|
| Arkansas River | | | | | |
| Above Santa Fe Avenue | 4,790.00[1] | 7,000 | 14,000 | 20,000 | 40,000 |
| Above Mouth of Fountain Creek | 4,790.00[1] | 7,000 | 14,000 | 20,000 | N/A |
| Downstream from Fountain Creek | 5,717.00[1] | 20,000 | 48,000 | 67,000 | 140,000 |
| Fountain Creek | | | | | |
| 3,500 Feet Upstream from State Highway 47 | 917.00 | N/A | N/A | 64,000 | 130,000 |
| At Mouth | 927.00 | N/A | N/A | 64,000 | 130,000 |
| University Park Tributary | | | | | |
| 1,930 Feet Upstream from Jerry Murphy Road | 0.89 | 450 | 1,010 | 1,370 | 4,100 |
| At Jerry Murphy Road | 1.25 | 610 | 1,430 | 1,970 | 6,100 |
| Wild Horse-Dry Creek | | | | | |
| 2,980 Feet Upstream from 24th Street | 74.58 | 5,600 | 13,400 | 18,500 | 37,500 |
| Downstream of Denver & Rio Grande Western Railroad | 86.80 | 5,700 | 14,000 | 19,500 | 39,500 |

[1] 4,670 square miles controlled by Pueblo Dam at River Mile 1,293.7

REVISED TO REFLECT LOMR DATED APR 25 1991

| FLOODING SOURCE | | FLOODWAY | | | BASE FLOOD WATER SURFACE ELEVATION | | | |
|---|---|---|---|---|---|---|---|---|
| CROSS SECTION | DISTANCE[1] | WIDTH (FEET) | SECTION AREA (SQUARE FEET) | MEAN VELOCITY (FEET PER SECOND) | REGULATORY (FEET NGVD) | WITHOUT FLOODWAY (FEET NGVD) | WITH FLOODWAY (FEET NGVD) | INCREASE (FEET NGVD) |
| Fountain Creek | | | | | | | | |
| L | 14,610 | 989 | 10,996 | 5.8 | 4,707.8 | 4,707.8 | 4,707.8 | 0.0 |
| M | 15,300 | 580 | 5,027 | 12.7 | 4,707.8 | 4,707.8 | 4,707.8 | 0.0 |
| N | 16,550 | 646 | 4,382 | 14.6 | 4,712.9 | 4,712.9 | 4,712.9 | 0.0 |
| O | 17,345 | 984 | 9,908 | 6.5 | 4,718.0 | 4,718.0 | 4,718.0 | 0.0 |
| P | 18,370 | 869 | 5,965 | 10.7 | 4,719.4 | 4,719.4 | 4,719.4 | 0.0 |
| Q | 20,740 | 866 | 7,912 | 8.1 | 4,733.4 | 4,733.4 | 4,733.4 | 0.0 |
| R | 21,715 | 762 | 6,973 | 9.2 | 4,741.9 | 4,741.9 | 4,741.9 | 0.0 |
| S | 22,650 | 1,093 | 8,376 | 7.6 | 4,744.6 | 4,744.6 | 4,744.6 | 0.0 |
| T | 23,510 | 1,071 | 5,787 | 11.1 | 4,747.1 | 4,747.1 | 4,747.1 | 0.0 |
| U | 24,910 | 720 | 4,519 | 14.2 | 4,754.4 | 4,754.4 | 4,754.4 | 0.0 |

REVISED TO
REFLECT LOMR
DATED APR 25 1991

[1]Feet Above Mouth (Along Profile Base Line)

TABLE 4

FEDERAL EMERGENCY MANAGEMENT AGENCY

**CITY OF PUEBLO, CO**
**(PUEBLO CO.)**

# FLOODWAY DATA

**FOUNTAIN CREEK**

| FLOODING SOURCE | PANEL[1] | ELEVATION DIFFERENCE[2] BETWEEN 1% (100 - YEAR) FLOOD AND 10% (10 - YEAR) | 2% (50 - YEAR) | 0.2% (500 - YEAR) | FLOOD HAZARD FACTOR | ZONE | BASE FLOOD ELEVATION (FEET NGVD)[3] |
|---|---|---|---|---|---|---|---|
| Fountain Creek | | | | | | | |
| Reach 7 | 0006,0010[4] | -7.1 | N/A | N/A | 070 | A14 | Varies - See Map |
| Reach 8 | 0006 | -7.4 | -2.0 | 8.4 | 075 | A15 | Varies - See Map |
| Reach 9 | 0003,0006 | -4.5 | -1.7 | 4.7 | 045 | A9 | Varies - See Map |

REVISED TO REFLECT LOMR DATED APR 25 1991

[1] Flood Insurance Rate Map Panel  [2] Weighted Average  [3] Rounded to Nearest Foot

[4] Reaches 1 through 6 have been deleted

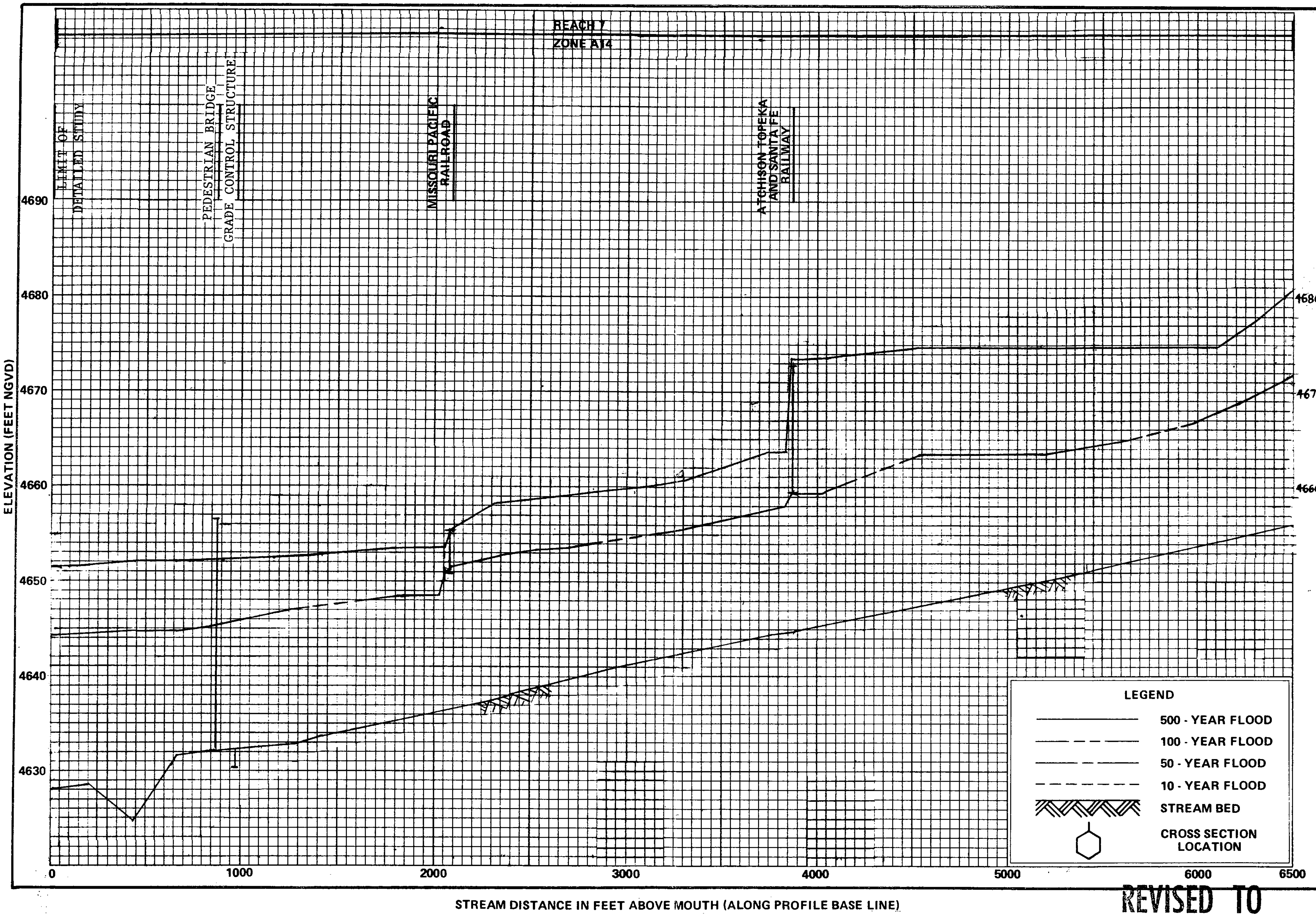
TABLE 5

FEDERAL EMERGENCY MANAGEMENT AGENCY

CITY OF PUEBLO, CO
(PUEBLO CO.)

# FLOOD INSURANCE ZONE DATA

## FOUNTAIN CREEK




FLOOD PROFILES

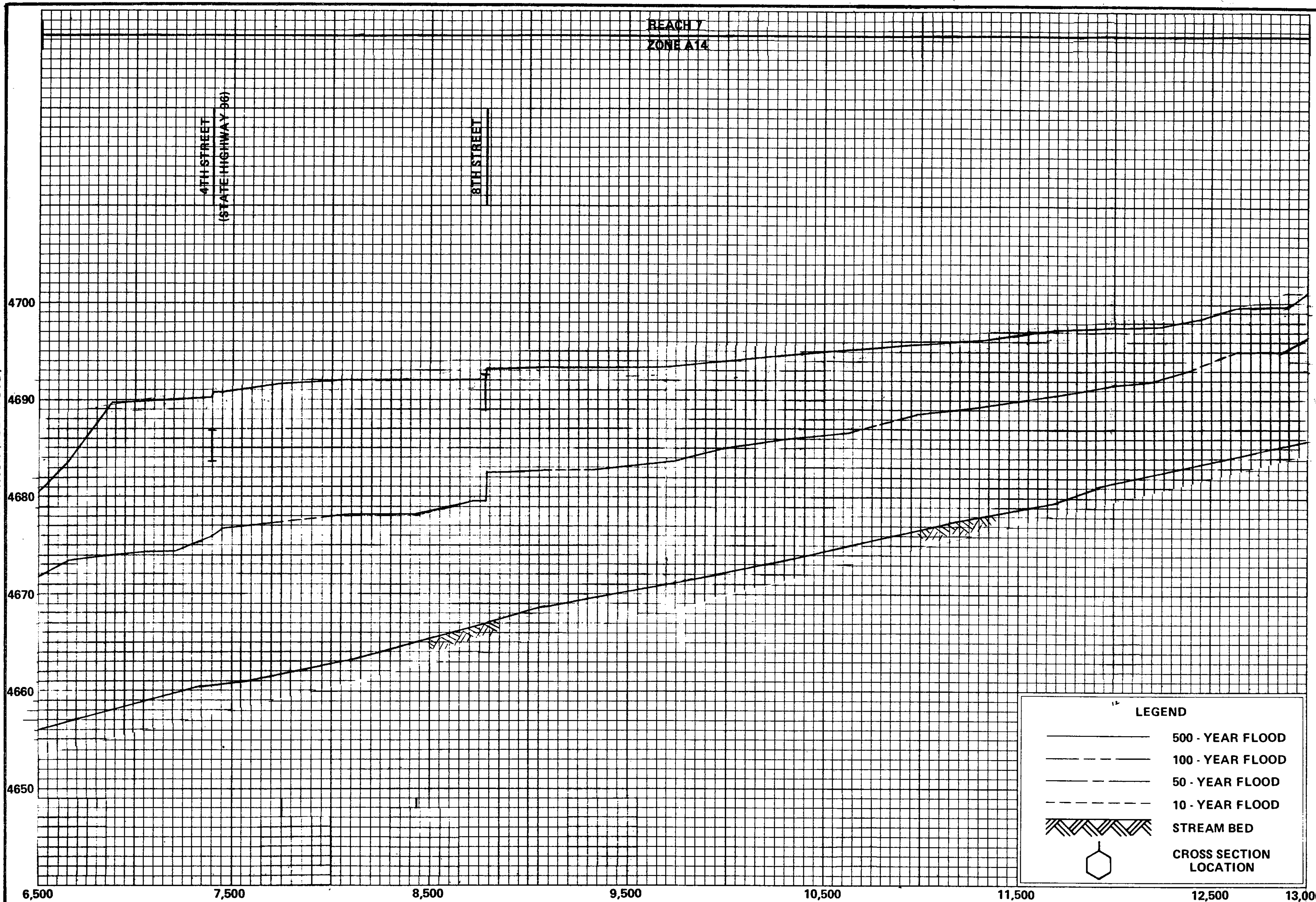
FOUNTAIN CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

CITY OF PUEBLO, CO
(PUEBLO CO.)

02P

REVISED TO REFLECT LOMR DATED APR 25 1991




STREAM DISTANCE IN FEET ABOVE MOUTH (ALONG PROFILE BASE LINE)

FLOOD PROFILES

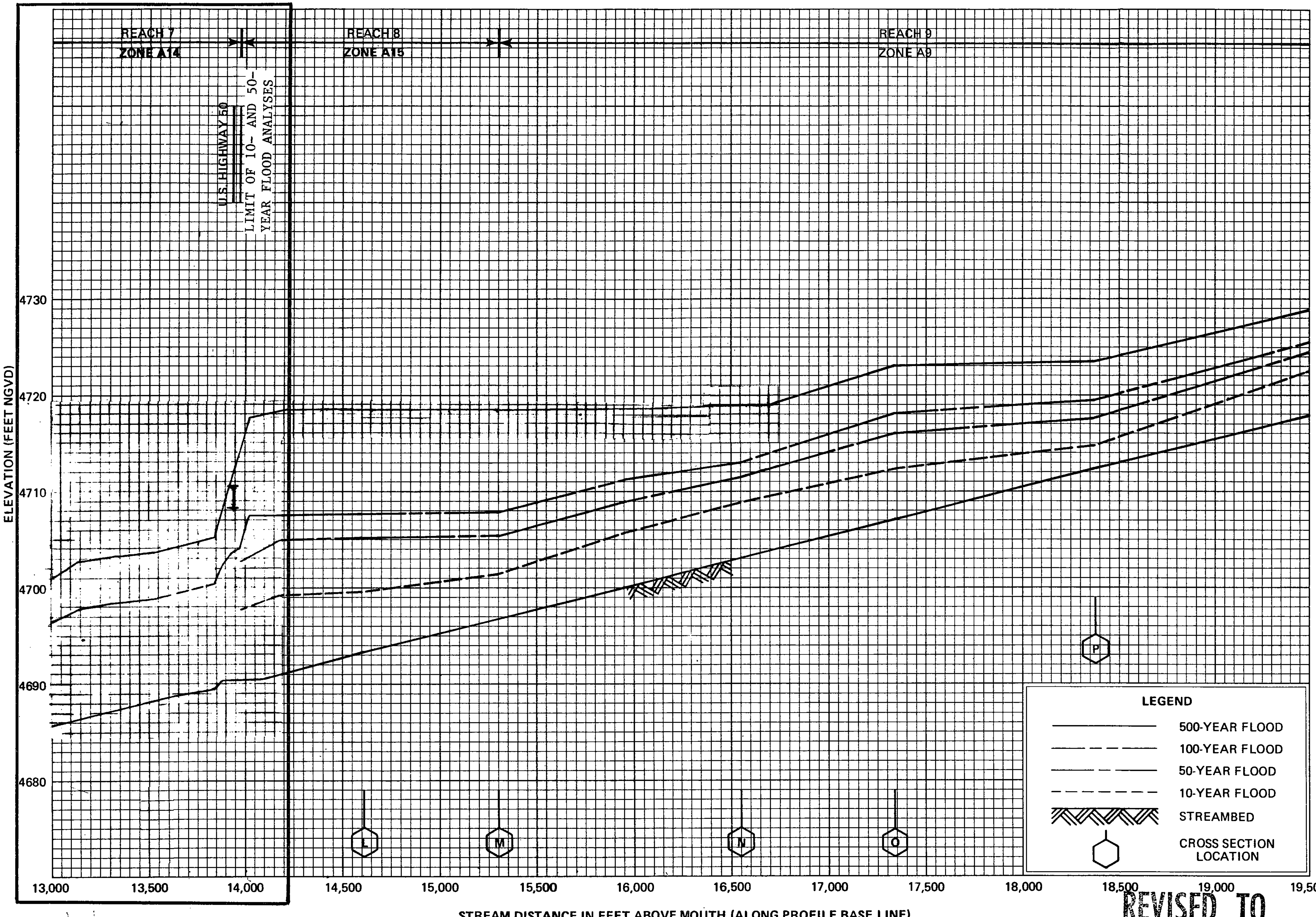
FOUNTAIN CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

CITY OF PUEBLO, CO
(PUEBLO CO.)

03P

REVISED TO REFLECT LOMR DATED APR 25 1991




FLOOD PROFILES

FOUNTAIN CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

CITY OF PUEBLO, CO
(PUEBLO CO.)

04P

REVISED TO REFLECT LOMR DATED APR 2 5 1991


ELEVATION REFERENCE MARKS

| REFERENCE MARK | ELEVATION (FT. NGVD) | DESCRIPTION OF LOCATION |
|---|---|---|
| RM7 | 4713.193 | U.S. Coast and Geodetic Survey disk stamped "D 4 1925 ELEV 4713.330 FT" located 3.0 miles north along the Denver and Rio Grande Western Railroad from the Union Station at Pueblo, 73.6 feet east of the southeast corner of the Standard Fire Brick Company office, 36.5 feet north of the centerline of East 24th Street, 40.6 feet southwest of the southwest rail of the main track, 17 feet south of a telephone pole, 7 feet southwest of the centerline of a track road, 1.9 feet south of a witness post, 0.4 mile south of milepost 116, set in the top of a concrete post which projects 0.9 foot above the ground. |
| RM8 | 4679.695 | U.S. Coast and Geodetic Survey disk stamped "Q 348 1953" located 1.7 miles north along the Denver and Rio Grande Western Railroad from the Union Station at Pueblo, 34.3 feet west of the west rail of the Denver and Rio Grande Western Railroad, 21.6 feet west of the west rail of the Atchison, Topeka and Santa Fe Railroad, 3 feet above the ground, at the 8th Street overpass, set vertically in the east side of the south pier of the 1st set of 2 piers west of the tracks. |

## KEY TO MAP

| | |
|---|---|
| 500-Year Flood Boundary | ZONE B |
| 100-Year Flood Boundary | |
| Zone Designations | ZONE A1 / ZONE A5 |
| 100-Year Flood Boundary | |
| 500-Year Flood Boundary | ZONE B |
| Base Flood Elevation Line With Elevation In Feet** | 513 |
| Base Flood Elevation in Feet Where Uniform Within Zone** | (EL 987) |
| Elevation Reference Mark | RM7× |
| Zone D Boundary | |
| River Mile | •M1.5 |

**Referenced to the National Geodetic Vertical Datum of 1929

## EXPLANATION OF ZONE DESIGNATIONS

| ZONE | EXPLANATION |
|---|---|
| A | Areas of 100-year flood; base flood elevations and flood hazard factors not determined. |
| A0 | Areas of 100-year shallow flooding where depths are between one (1) and three (3) feet; average depths of inundation are shown, but no flood hazard factors are determined. |
| AH | Areas of 100-year shallow flooding where depths are between one (1) and three (3) feet; base flood elevations are shown, but no flood hazard factors are determined. |
| A1-A30 | Areas of 100-year flood; base flood elevations and flood hazard factors determined. |
| A99 | Areas of 100-year flood to be protected by flood protection system under construction; base flood elevations and flood hazard factors not determined. |
| B | Areas between limits of the 100-year flood and 500-year flood; or certain areas subject to 100-year flooding with average depths less than one (1) foot or where the contributing drainage area is less than one square mile; or areas protected by levees from the base flood. (Medium shading) |
| C | Areas of minimal flooding. (No shading) |
| D | Areas of undetermined, but possible, flood hazards. |
| V | Areas of 100-year coastal flood with velocity (wave action); base flood elevations and flood hazard factors not determined. |
| V1-V30 | Areas of 100-year coastal flood with velocity (wave action); base flood elevations and flood hazard factors determined. |

## NOTES TO USER

This map is for use in administering the National Flood Insurance Program; it does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size, or all planimetric features outside Special Flood Hazard Areas.

Areas of special flood hazard (100-year flood) include Zones A, A1-30, AE, AH, AO, A99, V, V1-30 AND VE.

Certain areas not in the Special Flood Hazard Areas (zones A and V) may be protected by flood control structures.

Coastal base flood elevations apply only landward of the shoreline shown on this map.

For adjoining map panels, see separately printed Index to Map Panels.

INITIAL IDENTIFICATION:
AUGUST 24, 1973

FLOOD HAZARD BOUNDARY MAP REVISIONS:

FLOOD INSURANCE RATE MAP EFFECTIVE:
AUGUST 24, 1973

FLOOD INSURANCE RATE MAP REVISIONS:

Map revised July 1, 1974 to change zone designations.

Map revised February 27, 1976 to reflect curvilinear flood boundary and change community boundary and to add shallow flood hazard area.

Map revised April 2, 1976 to change zone designations and adjust curvilinear flood boundary.

Map revised September 29, 1986 to change flood plain boundaries, zone designations, base flood elevations, corporate limits, scale, cultural features, or map format.

To determine if flood insurance is available in this community, contact your insurance agent, or call the National Flood Insurance Program, at (800) 638-6620.

N

APPROXIMATE SCALE IN FEET
400 0 400

NATIONAL FLOOD INSURANCE PROGRAM

# FIRM
## FLOOD INSURANCE RATE MAP

CITY OF
PUEBLO,
COLORADO
PUEBLO COUNTY

PANEL 6 OF 15
(SEE MAP INDEX FOR PANELS NOT PRINTED)

REVISED TO
REFLECT LOMR
DATED APR 2 5 1991

COMMUNITY-PANEL NUMBER
085077 0006 C

MAP REVISED:
SEPTEMBER 29, 1986

Federal Emergency Management Agency


## ELEVATION REFERENCE MARKS

| REFERENCE MARK | ELEVATION (FT. NGVD) | DESCRIPTION OF LOCATION |
|---|---|---|
| RM7 | 4713.193 | U.S. Coast and Geodetic Survey disk stamped "D 4 1925 ELEV 4713.330 FT" located 3.0 miles north along the Denver and Rio Grande Western Railroad from the Union Station at Pueblo, 73.6 feet east of the southeast corner of the Standard Fire Brick Company office, 36.5 feet north of the centerline of East 24th Street, 40.6 feet southwest of the southwest rail of the main track, 17 feet south of a telephone pole, 7 feet southwest of the centerline of a track road, 1.9 feet south of a witness post, 0.4 mile south of milepost 116, set in the top of a concrete post which projects 0.9 foot above the ground. |
| RM8 | 4679.695 | U.S. Coast and Geodetic Survey disk stamped "Q 348 1953" located 1.7 miles north along the Denver and Rio Grande Western Railroad from the Union Station at Pueblo, 34.3 feet west of the west rail of the Denver and Rio Grande Western Railroad, 21.6 feet west of the west rail of the Atchison, Topeka and Santa Fe Railroad, 3 feet above the ground, at the 8th Street overpass, set vertically in the east side of the south pier of the 1st set of 2 piers west of the tracks. |

## KEY TO MAP

500-Year Flood Boundary
100-Year Flood Boundary
FLOODWAY FRINGE
FLOODWAY
100-Year Flood Boundary
500-Year Flood Boundary
Approximate 100-Year Flood Boundary
Cross Section Line
Elevation Reference Mark RM7
River Mile • M1.5

## NOTES TO USER

Boundaries of the floodways were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the Federal Emergency Management Agency.

This map was prepared to facilitate flood plain management activities only; it may not show all special flood hazard areas in the community or all planimetric features outside of the flood plain. Refer to the latest official Flood Insurance Rate Map for any additional areas of special flood hazard.

Floodway widths in some areas may be too narrow to show to scale. Refer to Floodway Data Table where floodway width is shown at 1/20 inch.

For adjoining map panels, see separately printed Index To Map Panels.

NATIONAL FLOOD INSURANCE PROGRAM

# FLOODWAY
## FLOOD BOUNDARY AND FLOODWAY MAP

CITY OF
PUEBLO,
COLORADO
PUEBLO COUNTY

PANEL 6 OF 15
(SEE MAP INDEX FOR PANELS NOT PRINTED)

REVISED TO REFLECT LOMR DATED APR 2 5 1991

COMMUNITY-PANEL NUMBER
085077 0006

MAP REVISED:
SEPTEMBER 29, 1986

Federal Emergency Management Agency