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SUMMARY

1. The subsurface conditions encountered in the borings consisted of overburden fills and natural soils extending to relatively shallow bedrock or to the full depth explored. The fill ranged from silty to clayey gravel to clayey sand with occasional sandy lean clay and isolated debris and organic material. The fill encountered in the borings appeared to be relatively dense; however, the lateral extent, depth and degree of compaction of the existing fill were not determined.

   The natural overburden soils generally consisted of poorly- to well-graded sand alluvium containing variable silt and gravel, occasional silty sand lenses, and isolated organic material. Based on sampler penetration resistance, the alluvial sands ranged from medium dense to dense. The bedrock consisted of very hard claystone with isolated lenses of sandy siltstone. River sediments generally consisted of fine- to coarse-grained, poorly-graded gravel with sand to well-graded sand with gravel, with occasional zones of silty clayey sand with gravel.

   Groundwater was encountered in one boring during drilling at a depth of about 9 feet. Stabilized groundwater levels were measured 12 days after drilling in all 4 borings at depths ranging from about 6 to 16.5 feet.

2. It should be feasible to support drop structures, ramp pavements, and shallow spread footings and mats directly on natural granular alluvial soils and on claystone bedrock, or on properly compacted structural fill extending to natural soils or claystone bedrock. Spread footing and mat foundations bearing on natural granular soils or claystone bedrock, or on properly compacted structural fill extending to natural granular soils or claystone bedrock, should be designed for a net allowable bearing pressure of 2,500 psf. The net allowable bearing pressure may be increased by one-third for transient loadings.

3. The existing overburden soils should be suitable for use as site grading fill, and some may suitable for use as structural fill beneath foundations and pavements and as retaining wall backfill. However, the claystone bedrock is generally very hard and will likely be difficult to break down and adequately process for use as site grading fill.

PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical engineering study for the proposed recreation and habitat improvements along the Overland Regional Park to Grant Frontier Park reach of the South Platte River in Denver, Colorado. The overall project site and individual project segments are shown on Figs. 1A through 1C. The study was performed in general accordance with the scope of work outlined in our Proposal No. P-12-379 to CDM Smith dated July 27, 2012.

A field exploration program consisting of exploratory borings and river sediment sampling was conducted to obtain information on subsurface conditions at specific locations along the project reach. Samples of the soils and bedrock obtained during the field exploration program were tested in our laboratory to determine their classification and engineering characteristics. The results of the field exploration and laboratory testing programs were analyzed to develop recommendations and construction considerations for the proposed recreation and habitat improvements.

This report has been prepared to summarize the data obtained during this study and to present our conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to construction of the proposed recreation and habitat improvements are included in the report.

PROPOSED CONSTRUCTION

We understand the project is part of the South Platte River Vision Implementation Plan developed by The Greenway Foundation and the City and County of Denver (City). The plan's objective is to improve currently limited river access and recreation opportunities, improve wildlife habitat, and provide a water quality benefit. Accomplishing those objectives will include providing multi-use river access facilities, channel bank modifications to provide additional riparian habitat such as pool and riffle areas, and modification of existing drop structures to provide boating and fishing experiences.

Along the alignment, improvements will include modifying the existing drop structure located just south of South Florida Avenue, bank regrading and vegetation removal in several areas, creation of two backwater/wetland areas along the east side of the river, construction of a water quality swale at the Harvard Gulch outfall, and construction of areas for accessing the river, including boat put in and take out locations. The project will include relocating existing concrete trail segments and park facilities in areas.

If the proposed construction varies significantly from that described above or depicted in this report, we should be notified to reevaluate the recommendations provided in this report.

SITE CONDITIONS

Boring 1 was drilled on the west side of the South Platte River just north of a pump station located at the lower end of a short gravel service road extending from Platte River Drive. Boring 1 was also located near the west end of the existing drop structure located just south of West Florida Avenue.

Borings 2 through 4 were drilled within Denver City parks located along the east side of the South Platte River. Borings 2 and 3 were drilled in Pasquinel's Landing Park and Grant Frontier Park, respectively, and were located a few feet off of the existing paved South Platte River trail. Both of these locations were relatively flat and a few feet above the river level. Boring 4 was drilled just north of the Harvard Gulch storm water outfall at a location approximately halfway between the South Platte River trail to the west and a paved alleyway to the east. Site grades at the location of Boring 4 slope gradually upward to the east from the South Platte River trail. Vegetation ranged from sod at the Borings 2 and 4 to native vegetation at Boring 3.

At the time of our field exploration program, the water level in the river was very low and significant portions of the riverbed were exposed. The exposed riverbed conditions consisted of a combination of scoured bedrock overlain in places by granular alluvium.

SUBSURFACE CONDITIONS

Subsurface Exploration Program: The subsurface conditions along the project reach were explored by drilling 4 exploratory borings to depths of about 20 feet below existing ground surface, and obtaining in-river sediment samples at three locations upstream of the existing drop structure at South Florida Avenue. The sediment samples were obtained by hand boring to depths ranging from about 3 to 7 feet below river bottom. The approximate locations of the exploratory borings and sediment samples are shown on Figs. 1A through 1C. Graphic logs of the borings and sediment samples are presented on Fig. 2, and a legend and notes describing the soils and bedrock encountered are presented on Fig. 3.

Subsurface Soil and Bedrock Conditions: The subsurface conditions encountered at the boring locations consisted of overburden fills and natural soils extending to relatively shallow bedrock in Borings 1, 3 and 4, and to the full depth explored of about 20 feet in Boring 2. The fill ranged from silty to clayey gravel in Boring 1 to clayey sand with occasional sandy lean clay in Borings 2 through 4. The fill encountered in Boring 1 was gray-brown, ranged from slightly moist to wet below groundwater, and contained asphalt and concrete debris and isolated organics. The predominantly clayey sand fill encountered in Borings 2 through 4 varied from brown to gray-brown, ranged from slightly moist to moist, and contained isolated gravel. The fill materials extended to bedrock at depths of about 8 and 5 feet in Borings 1 and 4, respectively. Based on sampler penetration resistance, the fill encountered in the borings appeared to be relatively dense; however, the lateral extent, depth and degree of compaction of the existing fill were not determined.

Natural overburden soils were encountered beneath the fill in Borings 2 and 3 and extended to the full depth explored of about 20 feet in Boring 2 and to bedrock at a depth of about 8 feet in Boring 3. The natural overburden soils encountered in the borings generally consisted of poorly- to well-graded sand alluvium containing variable silt and gravel, occasional silty sand lenses, and isolated organic material. The alluvial sands were typically brown and slightly moist to wet below groundwater. Based on sampler penetration resistance, the alluvial sands ranged from medium dense to dense.

The bedrock encountered in Borings 1, 3 and 4 consisted of blue-gray claystone with isolated lenses of olive-brown sandy siltstone. The bedrock was slightly moist and very hard, based on sampler penetration resistance.

The sediments encountered at the sample locations generally consisted of fine- to coarse-grained, poorly graded gravel with sand to well-graded sand with gravel, with occasional zones of silty clayey sand with gravel. The sediments were typically brown.

General Groundwater Conditions: Groundwater was encountered in Boring 2 during drilling at a depth of about 9 feet. Stabilized groundwater levels were measured 12 days after drilling in all 4 borings at depths ranging from about 6 to 16.5 feet below ground surface.

LABORATORY TESTING

Laboratory testing was performed on selected soil and bedrock samples obtained from the borings to determine in situ soil moisture content and dry density, Atterberg limits, swell-consolidation characteristics, and gradation. The results of the laboratory tests are shown to the right of the logs on Fig. 2 and summarized in Table I. The results of specific tests are graphically plotted on Figs. 4 through 9. The testing was conducted in general accordance with recognized test procedures, primarily those of the American Society for Testing of Materials (ASTM).

Swell-Consolidation: Swell-consolidation tests were conducted on selected samples of the claystone bedrock in order to determine their compressibility and swell characteristics under loading and when submerged in water. Each sample was prepared and placed in a confining ring between porous discs, subjected to a surcharge pressure of 1,000 psf, and allowed to consolidate before being submerged. The sample height was monitored until deformation practically ceased under each load increment.

Results of the swell-consolidation tests are presented on Figs. 4 and 5 as curves of the final strain at each increment of pressure against the log of the pressure. The results of the swell-consolidation tests indicated nil to low swell potential or a slight tendency for additional compression when wetted under a surcharge pressure of 1,000 psf. The additional compression exhibited by one of the tested samples is possibly due to sample disturbance.

Index Properties: Samples were classified into categories of similar engineering properties in general accordance with the Unified Soil Classification System. This system is based on index properties, including liquid limit and plasticity index and grain size distribution. Values for moisture content, dry density, liquid limit and plasticity index, and the percent of soil passing the U.S. No. 4 and 200 sieves are presented in Table I and adjacent to the corresponding sample on the boring logs. Grain size distribution curves are presented on Figs. 6 through 9.

GEOTECHNICAL ENGINEERING CONSIDERATIONS

Subsurface conditions at planned site finished grades and at foundation level for the various anticipated improvements are expected to consist of relatively dense existing fill and predominantly granular natural soils or very hard claystone bedrock. Based on the results of the laboratory swell-consolidation testing, the claystone bedrock exhibited nil to very low swell potential when wetted at a surcharge pressure of 1,000 psf. The foundation levels of drop structures and other in-stream improvements are expected to be at or below ground water in places.

Considering the subsurface conditions encountered in the exploratory borings and the nature of the proposed improvements, it should be feasible to support drop structures, ramp pavements, and shallow spread footings and mats directly on natural granular alluvial soils and on claystone bedrock, or on properly compacted structural fill extending to natural soils or claystone bedrock.

The existing overburden soils should be suitable for use as site grading fill, and some may suitable for use as structural fill beneath foundations and pavements and as retaining wall backfill. However, the claystone bedrock is generally very hard and will likely be difficult to break down and adequately process for use as site grading fill.

Spread Footing and Mat Foundations: The design and construction criteria presented below should be observed for spread footing and mat foundation systems. The construction details should be considered when preparing project documents.

1. Spread footing and mat foundations bearing on natural granular soils or claystone bedrock, or on properly compacted structural fill extending to natural granular soils or claystone bedrock, should be designed for a net allowable bearing pressure of 2,500 psf. The net allowable bearing pressure may be increased by one-third for transient loadings.

2. Based on experience, we estimate total and settlement for spread footing and mat foundations designed and constructed as discussed in this section will be less than 1 inch. Differential settlement between similarly-loaded spread footing foundation elements or across a mat foundation should be less than ½ to ¾ of the total settlement. Non-uniformity of the subsurface conditions will contribute to total and differential settlements. Due to the nature of the construction, and anticipated relatively light net

foundation loads, much of the settlement should occur during construction upon initial loading of the foundations.

3. Spread footings should have a minimum footing width of 18 inches for continuous footings and a minimum width of 24 inches for isolated pad footings.

4. Footings and mats should be provided with adequate soil cover above their bearing elevation for frost protection. Placement of the bottom of footings and mats at least 36 inches below the exterior grade is typically used in this area.

5. Areas of existing fill encountered within the foundation excavation, or materials disturbed during excavation, should be removed and the foundations extended to adequate natural bearing material. As an alternate, the existing fill or disturbed materials may be removed and replaced with on-site or imported structural fill. New structural fill should extend down and out from the edges of the foundations at a 1 horizontal to 1 vertical projection.

6. Structural fill should consist of a non-expansive material with a swell potential that does not exceed 1% when remolded to 95% of the standard Proctor (ASTM D 698) maximum dry density at optimum moisture content and wetted under a surcharge pressure of 200 psf. Evaluation of potential sources may require determination of laboratory moisture-density relationships and swell consolidation tests on remolded samples. Structural fill should be placed and compacted to at least 95% of the standard Proctor (ASTM D 698) maximum dry density at moisture contents within 2 percentage points of the optimum moisture content for granular materials and between 0 and +3 percentage points of the optimum moisture content for clay materials.

7. It will be important to use proper construction equipment and techniques to avoid excessive disturbance of underlying wet subgrade soils. Use of low ground pressure tracked equipment or a hydraulic excavator working from outside of excavated areas may be necessary to avoid excessive subgrade disturbance during backfilling. Foundation excavations should also be completely dewatered such that foundation construction can be completed under relatively dry conditions.

8. Structural fill placed against the sides of the foundations to resist lateral loads should consist of on-site or imported material meeting the material and placement criteria recommended in Item 6.

9. The lateral resistance of a foundation placed on natural soils or compacted structural fill will be a combination of the sliding resistance of the foundation on the foundation materials and passive earth pressure against the side of the foundation. Resistance to sliding at the bottoms of the foundations can be calculated based on a coefficient of friction of 0.3. Passive pressure against the sides of the foundations can be calculated using a unit weight of 175 pcf for backfill zones that will not be inundated. Where backfill zones will be inundated at times, the passive pressure will need to be reduced based on the design water level. These lateral resistance values are working values.

10. A representative of the geotechnical engineer should observe all foundation excavations, observe and test compaction, and evaluate the suitability of all structural fill.

RETAINING STRUCTURES

Retaining structures should be designed for the lateral earth pressure generated by the backfill, which is a function of the degree of rigidity of the wall and the type of backfill material used. Below ground walls and retaining structures that are laterally supported and can be expected to undergo only a moderate amount of deflection should be designed for earth pressures based on the following equivalent fluid pressures:

CDOT Class 1 backfill (<20% passing No. 200 sieve) .... 50 pcf
On-site clayey sand backfill .... 55 pcf

Cantilevered retaining structures that can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for earth pressures based on the following equivalent fluid pressures:

CDOT Class 1 backfill (<20% passing No. 200 sieve) .... 40 pcf
On-site clayey sand backfill .... 45 pcf

The equivalent fluid pressure values recommended above assume drained conditions behind the retaining structures. The buildup of water behind a retaining structure will increase the lateral earth pressure imposed on the retaining structure. Laterally restrained retaining structures extending below the water table should be designed for the undrained at-rest condition using the following equivalent fluid pressures:

CDOT Class 1 backfill (<20% passing No. 200 sieve) .... 90 pcf
On-site clayey sand backfill .... 95 pcf

All of the equivalent fluid pressure values recommended above assume a horizontal backfill surface. An upward sloping backfill surface will increase the lateral earth pressure imposed on the retaining structure. Retaining structures should also be designed for appropriate surcharge pressures due to adjacent structures, vehicle traffic, and construction activities.

The zone of backfill placed behind retaining structures to within 2 feet of the ground surface should be sloped upward from the base of the structure at an angle of no steeper than 45 degrees from horizontal. The upper 2 feet of the structure backfill should consist of a relatively impervious on-site soil or a slab or pavement structure to reduce surface water infiltration into the backfill. To limit post-construction settlement, backfill should be placed in uniform lifts and compacted to at least 95% of the standard Proctor (ASTM D 698) maximum dry density where placed to depths of less than 8 feet and to at least 98% of the standard Proctor maximum dry density where placed at depths greater 8 feet. Granular backfill should be placed in 8-inch lifts and compacted at moisture contents within 2 percentage points of optimum. Care should be taken not to over compact the backfill since this could cause excessive lateral pressure on the walls. Hand compaction procedures, if necessary, should be used to prevent lateral pressures from exceeding the design values.

The lateral resistance of retaining structure foundations should be evaluated using foundation criteria presented in the "Spread Footing and Mat Foundations" section of this report. Backfill placed against the sides of the retaining structure footings to resist lateral loads should be compacted according to the criteria for backfill placed behind retaining structures presented above.

EXCAVATION AND GRADING CONSIDERATIONS

Site Preparation: Site preparation within areas to be graded should include stripping existing vegetation and topsoil, and excavation to establish planned site grades and subgrades for structures and other improvements. Material types encountered during site grading will generally consist of predominantly granular fills and natural soils and claystone bedrock. Although these materials can generally be excavated during site grading operations with heavy-duty conventional earth moving equipment, the contractor should be aware that the claystone bedrock is very hard and may be difficult to excavate in places.

Fill Materials: On-site granular fills and natural soils should be suitable for use as site grading fill and possibly as structural fill beneath foundations, ramp pavements, and flatwork areas. On-site natural clay soils, if encountered, and excavated claystone will not be suitable for use as structural fill beneath foundations, ramp pavements, and flatwork areas but may be suitable for use as site grading fill provided these materials can be adequately processed and compacted. Processing on-site clay soils and claystone would include moisture-conditioning to moisture contents between -1 and 3 percentage points above optimum and compacting these materials to at least 95 percent of the standard Proctor (ASTM D 698) maximum dry density where 8 feet thick or less, and to at least 100 percent of the standard Proctor (ASTM D 698) maximum dry density where thicker than 8 feet.

The contractor should provide adequate equipment to produce a uniform and stable fill that meets compaction and moisture requirements outlined in the pertinent sections of this report. As mentioned, the claystone bedrock is very hard. Based on our experience with similar bedrock materials, we believe it will be difficult for the Contractor to break down and moisture condition excavated bedrock sufficiently to meet the placement and compaction requirements recommended herein.

Temporary Excavations: We assume that the site excavations will be constructed by generally over-excavating the side slopes to a stable configuration where enough space is available. All excavations should be constructed in accordance with OSHA requirements, as well as state, local and other applicable requirements. Site excavations will generally encounter predominantly granular fills and natural soils and claystone bedrock. Fills and natural granular soils generally classify as OSHA Type C soils.

Claystone bedrock generally classifies as Type A soil, although fractured and weathered bedrock may classify as Type B. Claystone bedrock may also behave as a Type B soil if the bedrock slakes or deteriorates; this may occur if the claystone is left exposed for a significant period of time. The Contractor should consider these possibilities when planning excavations into bedrock.

Excavation below groundwater and/or the presence of ground water in excavated slopes may require slopes flatter than those recommended by OSHA and/or require temporary shoring. Excavations within the river bed will likely require temporary cofferdams to reduce seepage into the excavation. Cofferdam systems commonly used in the Denver area include interlocking driven sheet piles and earthen berms. Due to the very hard bedrock anticipated to underlie most of the project reach, installation of sheet piles to the depths adequate for stability may be very difficult.

Excavated slopes in fill and natural granular soils, and in claystone bedrock, may soften due to construction traffic and erode from surface runoff. Measures to keep surface runoff from excavation slopes, including diversion berms, should be considered.

Excavation Dewatering: Site excavations are likely to encounter groundwater, and excavations within the river bed may be below the water level in the river. In areas underlain by shallow bedrock, groundwater is likely to be perched in the soils overlying the bedrock and/or in fractured zones within the near surface portion of the bedrock, and excavations within the river bed may be subject to seepage through the cofferdams.

Where the groundwater table is encountered at or near the base of the excavation or seepage occurs through the cofferdams, we believe site excavations generally can be dewatered during construction using perimeter and lateral trenches combined with sumps. The trenches should be sloped to sumps where water can be pumped from the excavation. If pumping from sumps cannot handle groundwater infiltration or seepage through the cofferdams, more extensive dewatering systems may be required, such as wells or well points, or seepage cutoffs may need to be constructed. Dewatering using wells and/or well point systems would be anticipated for excavations extending a few feet below groundwater in the natural granular alluvial soils, which are anticipated to have relatively high permeability.

In general, we recommend that the groundwater level be maintained at least three feet below the bottom of foundation excavations at all times to mitigate disturbance of the foundation soils and/or bedrock and to facilitate placement and compaction of structural fill. This criterion is more applicable for excavations underlain by natural soils; subgrade stability and/or disturbance is anticipated to be less of a concern where the excavation subgrade consists of the very hard claystone bedrock.

<u>Permanent Cut and Fill Slopes</u>: Based on our experience with soils and bedrock similar to those encountered on the site, we recommend that unreinforced embankment fills and permanent cut slopes above the groundwater table be constructed no steeper than 2H:1V based on stability requirements and 3H:1V for reducing erosion susceptibility. No formal stability analyses were performed to evaluate the slopes recommended above. Published literature and our experience with similar cuts and fills indicate the recommended slopes should have adequate factors of safety.

Seepage may be encountered in permanent excavation slopes. The risk of slope instability will be significantly increased if seepage is encountered in cuts, and a stability investigation should be conducted to determine if the seepage will adversely affect permanent cuts and fills.

Slopes constructed of or excavated in on-site fills and natural soils are expected to be moderately to highly susceptible to surface erosion under moderate sheet flows and highly susceptible to erosion under concentrated flows. Susceptibility to erosion can be limited by constructing the slopes at flatter inclinations, as recommended above, by establishing an appropriate vegetative cover or providing appropriate erosion protection, and by providing good surface drainage to direct surface runoff away from the slope faces. Consideration should be given to armoring river bank slopes in areas that might be subjected to disturbed flows or higher flow velocities, such as the outside of channel bends or at channel structures.

To provide a uniform base for fill placement, the ground surface underlying all new fills should be carefully prepared by removing all organic matter, scarifying where feasible to a depth of 12 inches, and re-compacting to at least 95% of the standard Proctor maximum dry density at moisture contents within 2 percentage points of optimum. Fills should be benched into cuts or natural slopes exceeding 4H:1V. Vertical bench heights should be between 2 and 4 feet.

DESIGN AND CONSTRUCTION SUPPORT SERVICES

Kumar & Associates, Inc. should be retained to review the project plans and specifications so that comments can be made regarding interpretation and implementation of our geotechnical engineering recommendations before these contract documents are finalized. We are also available to assist the design team in preparing specifications for geotechnical aspects of the project, and performing additional studies if necessary to accommodate possible changes in the proposed construction after the completion of our study. We recommend that Kumar & Associates, Inc. be retained to provide observation and testing services during construction to document that the intent of this report and the requirements of the plans and specifications are being followed during construction, and to identify possible variations in foundation conditions from that encountered in this study so that we can re-evaluate our recommendations if needed.

LIMITATIONS

This study has been conducted in accordance with generally accepted geotechnical engineering practice in this area for exclusive use of the client for design purposes. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings at the locations indicated on Figs. 1A through 1C, and the proposed type of construction. This report does not reflect subsurface variations that may occur between the exploratory borings, and the nature and extent of variations across the site may not become evident until site grading and excavations are performed. If during construction, fill, soil, rock or water conditions appear to be different from those described herein, Kumar & Associates, Inc. should be advised at once so that a re-evaluation of the recommendations presented in this report can be made. Kumar & Associates, Inc. is not responsible for liability associated with interpretation of subsurface data by others.

The scope of services for this project does not include any environmental assessment of the site or identification of contaminated or hazardous materials or conditions. If the owner is concerned about the potential for such contamination, other studies should be undertaken.

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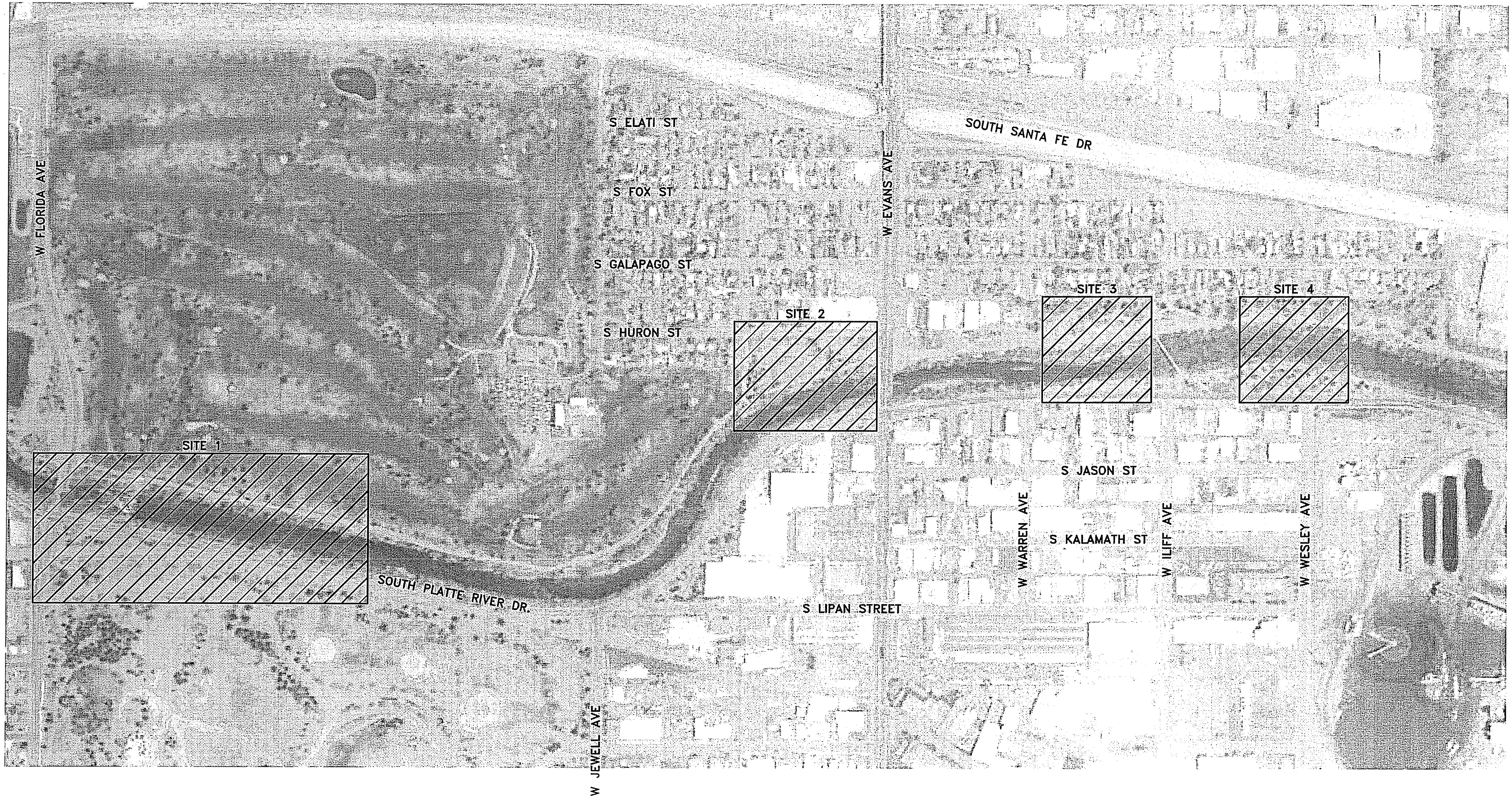

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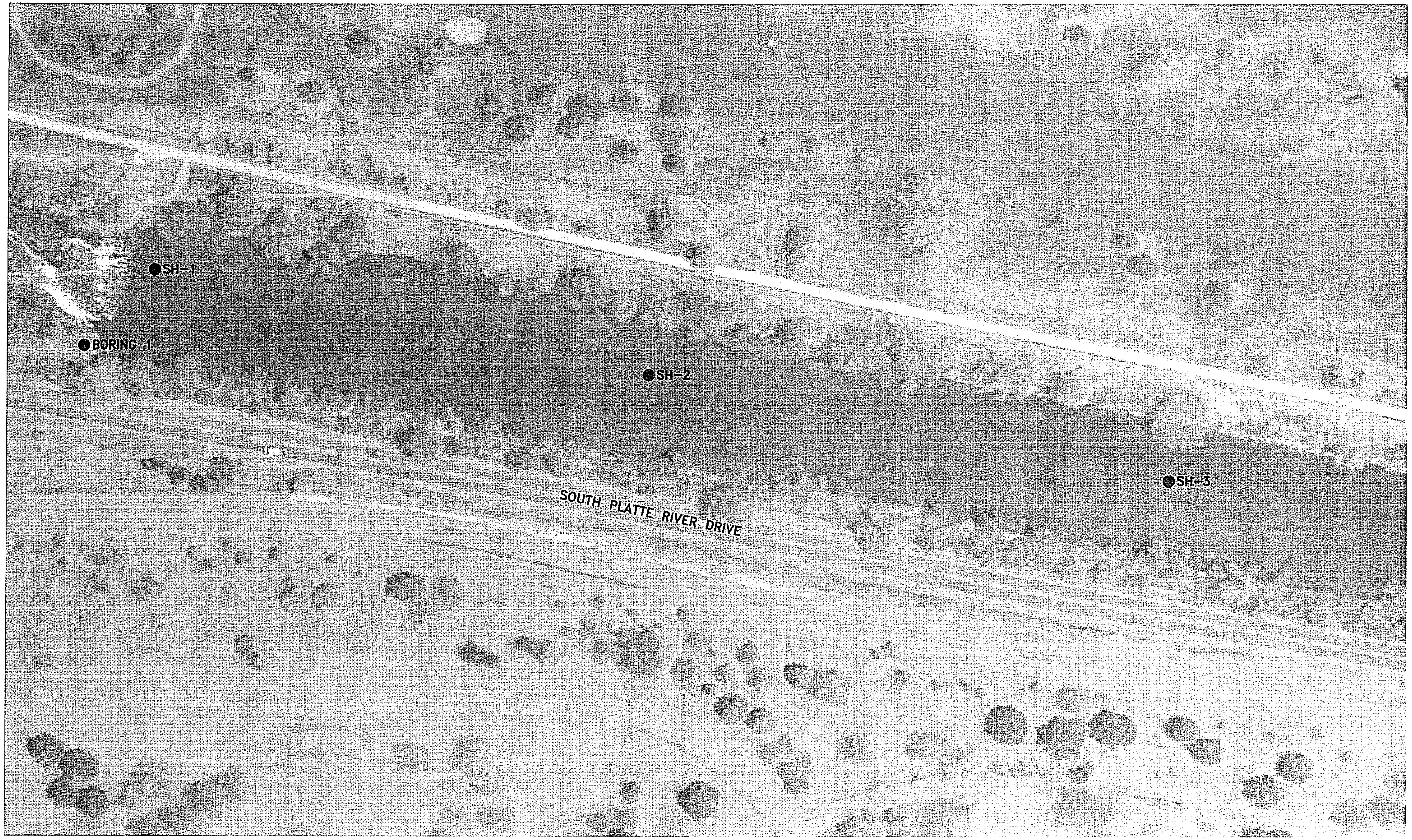
12-1-339	Kumar & Associates	SOUTH PLATTE RIVER RECREATION AND HABITAT IMPROVEMENTS, OVERTON POND PARK TO GRANT FRONTIER PARK REACH, DENVER, CO	SITE LOCATION	Fig. 1A



SITE 1

50 0 50 100
APPROXIMATE SCALE–FEET

12–1–339	Kumar & Associates	SOUTH PLATTE RIVER RECREATION AND HABITAT IMPROVEMENTS, OVERTON POND PARK TO GRANT FRONTIER PARK REACH, DENVER, CO	LOCATIONS OF EXPLORATORY BORINGS	Fig. 1B

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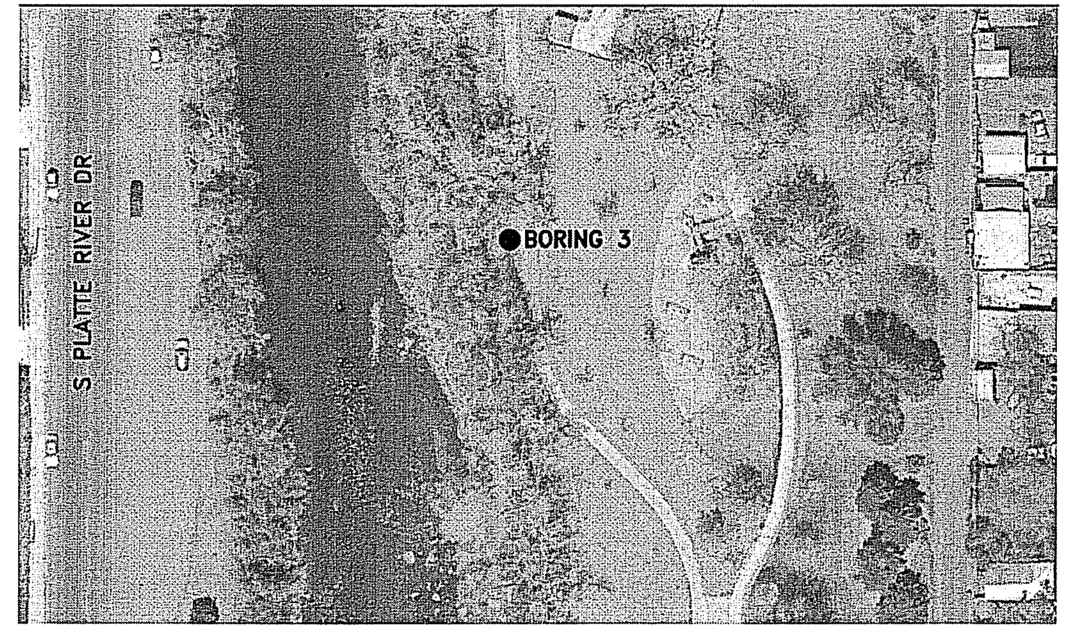
SITE 2: PASQUINEL'S LANDING PARK



SITE 3: GRANT FRONTIER PARK

SITE 4: HARVARD GULCH

50 0 50 100

APPROXIMATE SCALE–FEET

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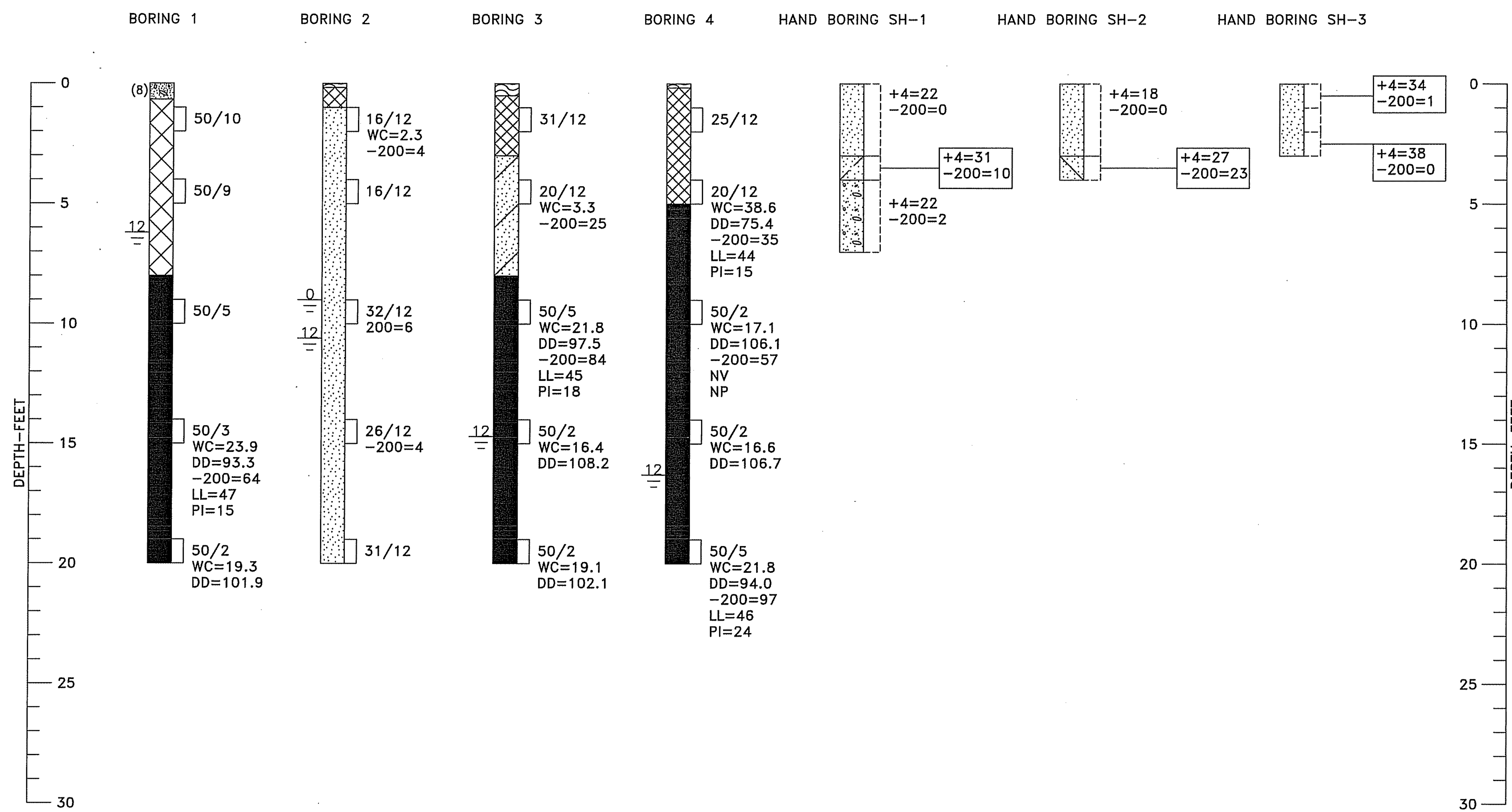
12–1–339	Kumar & Associates	SOUTH PLATTE RIVER RECREATION AND HABITAT IMPROVEMENTS, OVERTON POND PARK TO GRANT FRONTIER PARK REACH, DENVER, CO	LOCATIONS OF EXPLORATORY BORINGS	Fig. 1C



12-1-339	Kumar & Associates	SOUTH PLATTE RIVER RECREATION AND HABITAT IMPROVEMENTS, OVERTON POND PARK TO GRANT FRONTIER PARK REACH, DENVER, CO	LOGS OF EXPLORATORY BORINGS	Fig. 2

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LEGEND

TOPSOIL.

(8) BASE COURSE, THICKNESS IN INCHES SHOWN IN PARENTHESES TO LEFT OF THE LOG.

FILL: SILTY TO CLAYEY GRAVEL WITH SAND (GM–GC), DEBRIS (ASPHALT AND CONCRETE), AND ISOLATED ORGANICS, SLIGHTLY MOIST TO WET BELOW GROUND WATER, GRAY–BROWN.

FILL: CLAYEY SAND (SC) TO OCCASIONALLY SANDY LEAN CLAY (CL), FINE TO COARSE SAND, ISOLATED GRAVEL, SLIGHTLY MOIST TO MOIST, BROWN TO GRAY–BROWN.

POORLY– TO WELL–GRADED SAND WITH VARIABLE SILT AND GRAVEL (SP/SW), MEDIUM DENSE TO DENSE, SLIGHTLY MOIST TO WET BELOW GROUND WATER, BROWN.

POORLY–GRADED SAND WITH SILT (SP–SM), GRAVEL, SILTY SAND (SM) LENSES AND ISOLATED ORGANICS, MEDIUM DENSE, SLIGHTLY MOIST, BROWN.

WELL–GRADED SAND WITH SILTY CLAY AND GRAVEL (SW–SC), WET, BROWN.

POORLY–GRADED GRAVEL WITH SAND (GP), WET, BROWN.

SILTY CLAYEY SAND WITH GRAVEL (SM–SC), WET, BROWN.

CLAYSTONE BEDROCK WITH ISOLATED LENSES OF SANDY SILTSTONE BEDROCK, VERY HARD, SLIGHTLY MOIST, BLUE–GRAY (CLAYSTONE) TO OLIVE–BROWN (SILTSTONE).

DRIVE SAMPLE, 2–INCH I.D. CALIFORNIA LINER SAMPLE.

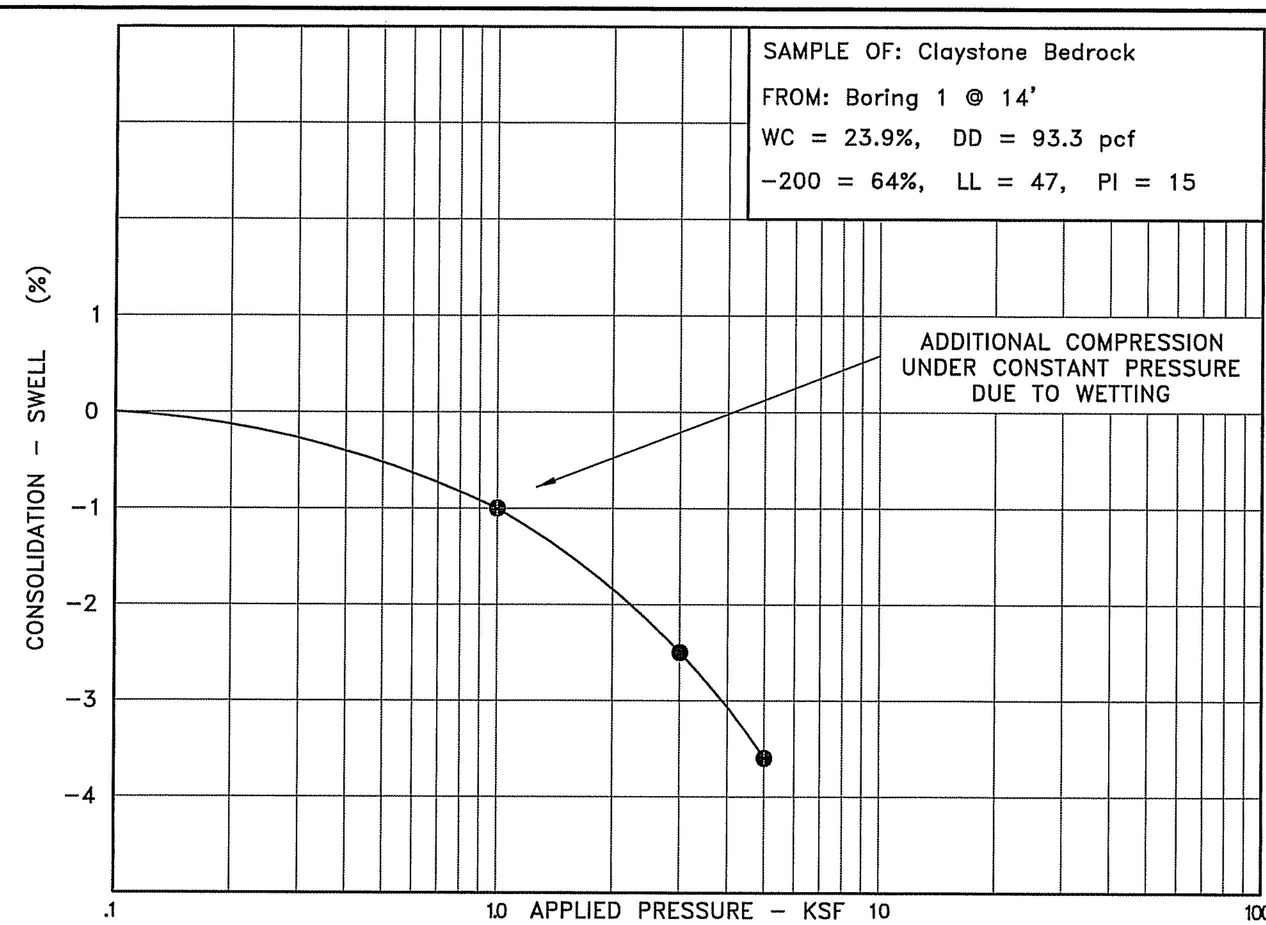
50/10 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 50 BLOWS OF A 140–POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 10 INCHES.

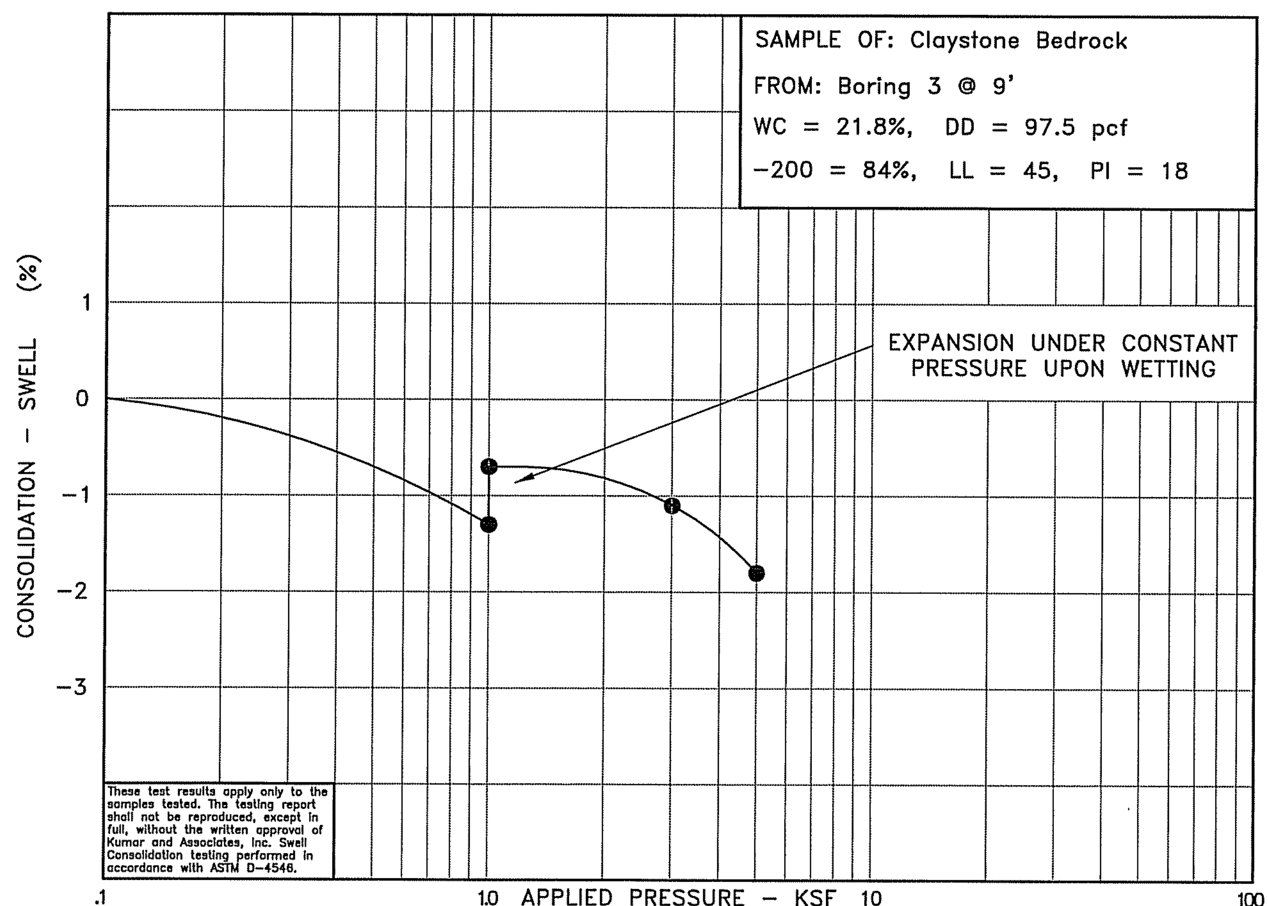
DISTURBED BULK SAMPLE.

12 DEPTH TO WATER LEVEL AND NUMBER OF DAYS AFTER DRILLING MEASUREMENT WAS MADE.

NOTES

1. EXPLORATORY BORINGS 1 THROUGH 4 WERE DRILLED ON JULY 20, 2012 WITH 4–INCH DIAMETER SOLID–STEM AND 7–INCH DIAMETER HOLLOW–STEM CONTINUOUS FLIGHT POWER AUGERS. SEDIMENT BORINGS SH–1 THROUGH SH–3 WERE HAND–AUGERED ON JULY 23, 2012.
2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE MEASURED APPROXIMATELY BY PACING FROM FEATURES SHOWN ON THE SITE PLAN PROVIDED.
3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE NOT MEASURED AND THE LOGS OF THE EXPLORATORY BORINGS ARE PLOTTED TO DEPTH.
4. THE EXPLORATORY BORING LOCATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.
5. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.
6. GROUND WATER LEVELS SHOWN ON THE LOGS WERE MEASURED AT THE TIME AND UNDER CONDITIONS INDICATED. FLUCTUATIONS IN THE WATER LEVEL MAY OCCUR WITH TIME.
7. LABORATORY TEST RESULTS:
   WC = WATER CONTENT (%) (ASTM D 2216);
   DD = DRY DENSITY (pcf) (ASTM D 2216);
   +4 = PERCENTAGE RETAINED ON NO. 4 SIEVE (ASTM D 422);
   –200 = PERCENTAGE PASSING NO. 200 SIEVE (ASTM D 1140);
   LL = LIQUID LIMIT (ASTM D 4318);
   PI = PLASTICITY INDEX (ASTM D 4318);
   NP = NON–PLASTIC (ASTM D 4318);
   NV = NO LIQUID LIMIT VALUE (ASTM D 4318).





12-1-339	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 4

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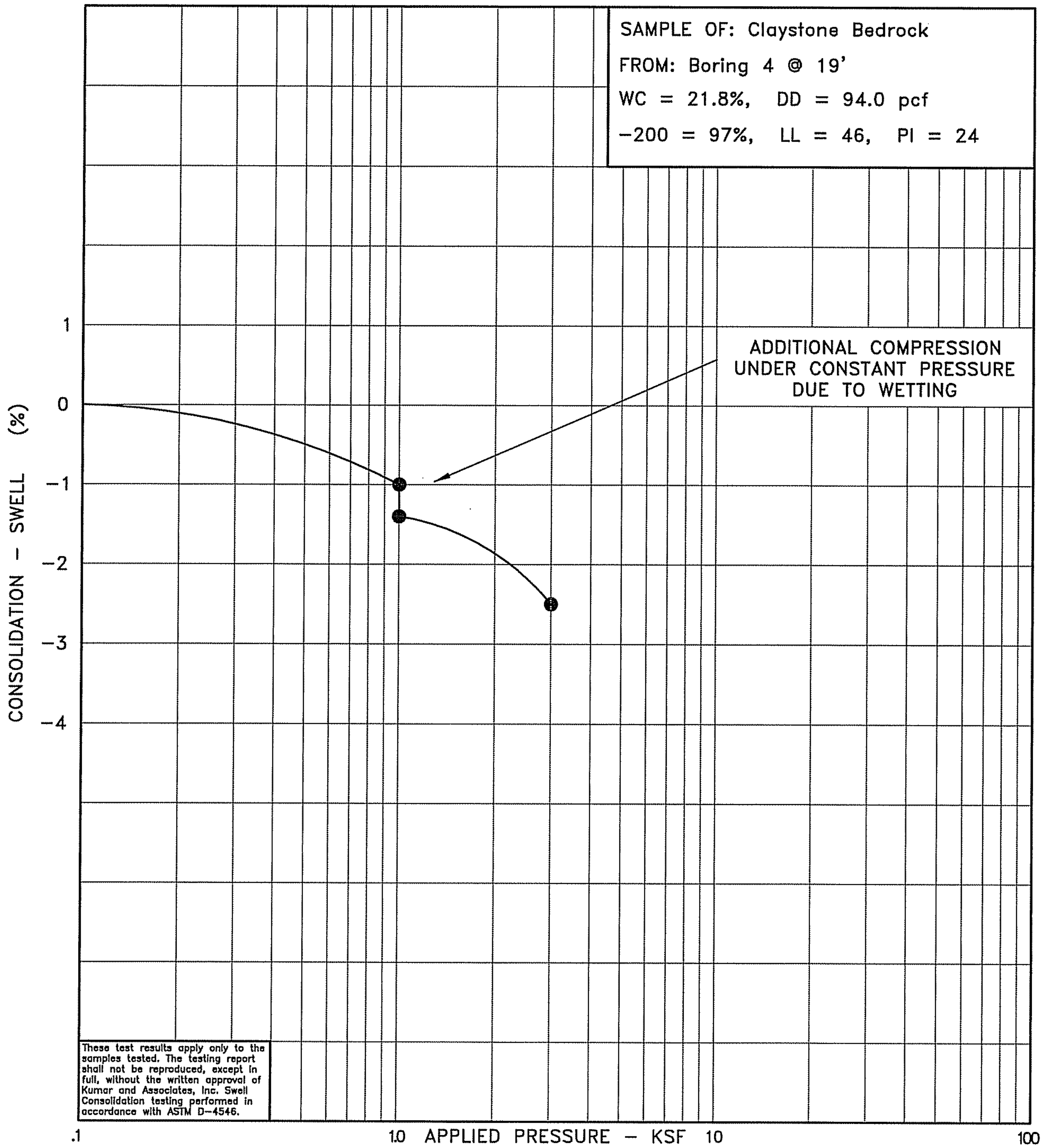

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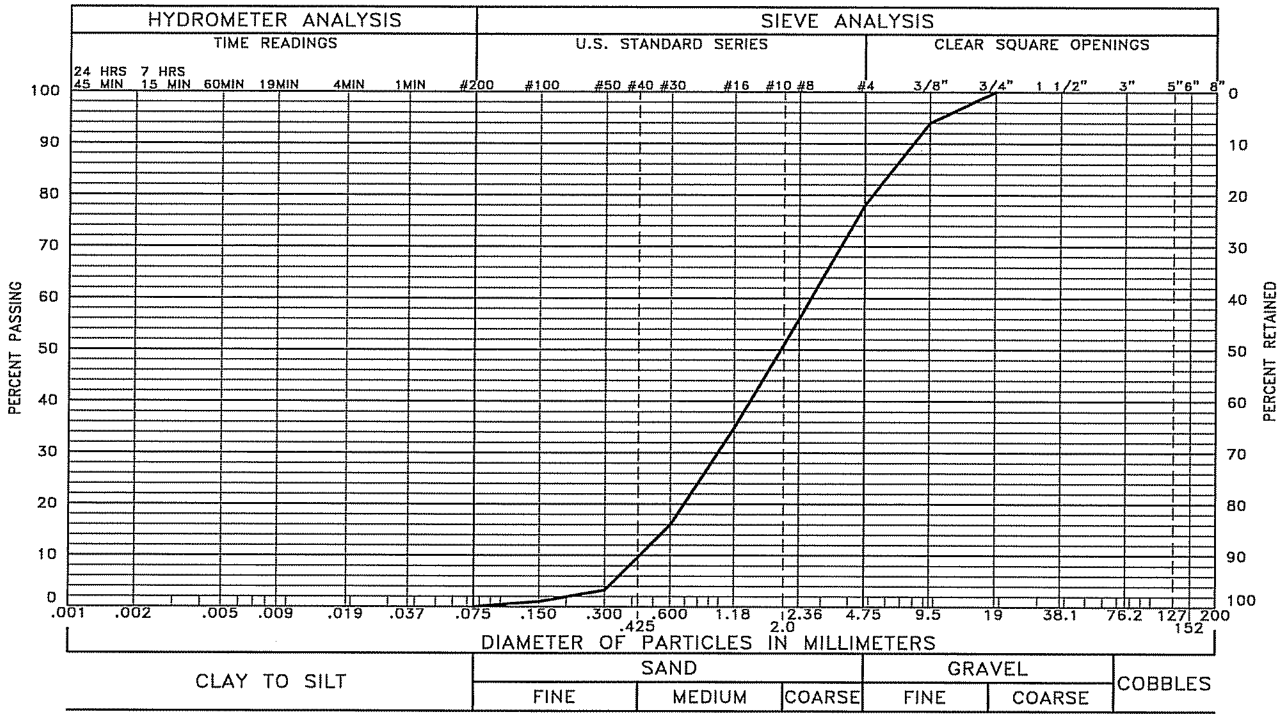
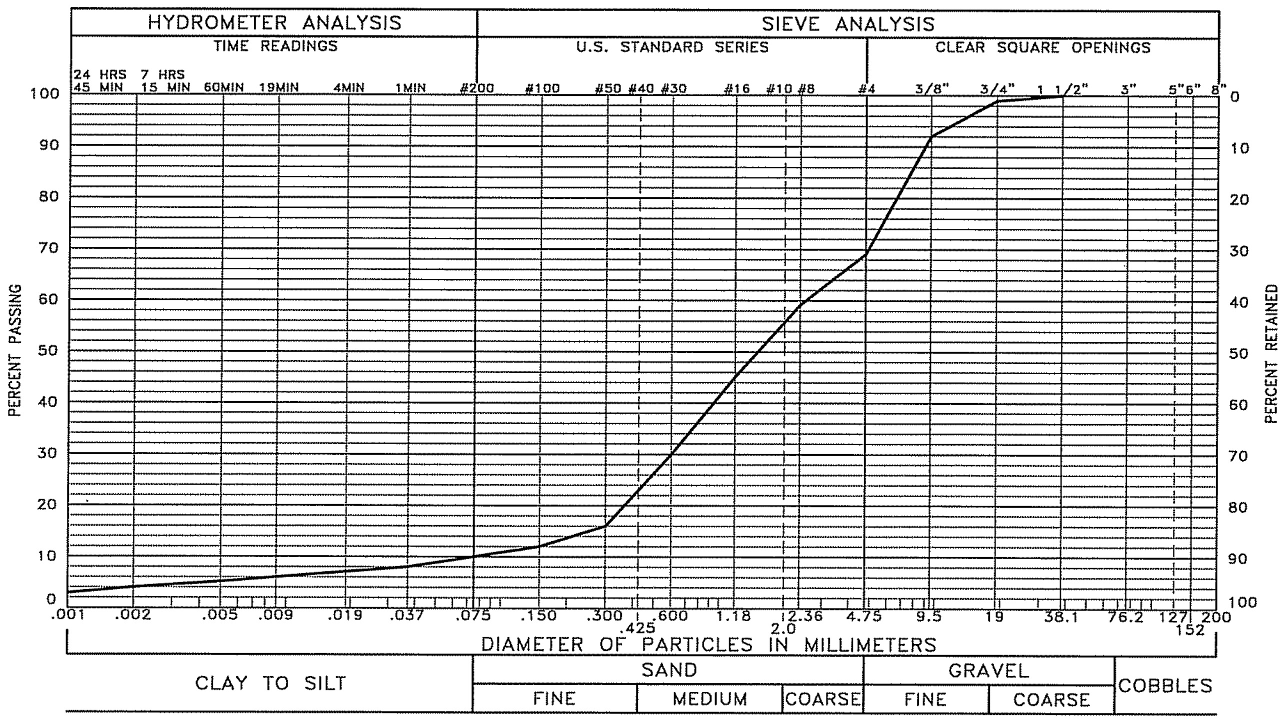
12-1-339	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 5



GRAVEL 22 % SAND 78 % SILT AND CLAY 0 %

LIQUID LIMIT PLASTICITY INDEX

SAMPLE OF: Poorly-Graded Sand with Gravel (SP) FROM: Boring SH-1 @ 0-3'



GRAVEL 31 % SAND 59 % SILT AND CLAY 10 %

LIQUID LIMIT PLASTICITY INDEX

SAMPLE OF: Well Graded Sand with Silty Clay (SW-SC) FROM: Boring SH-1 @ 3'-4'

These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. Sieve analysis testing is performed in accordance with ASTM D422, ASTM C136 and/or ASTM D1140.

12-1-339	Kumar & Associates	GRADATION TEST RESULTS	Fig. 6

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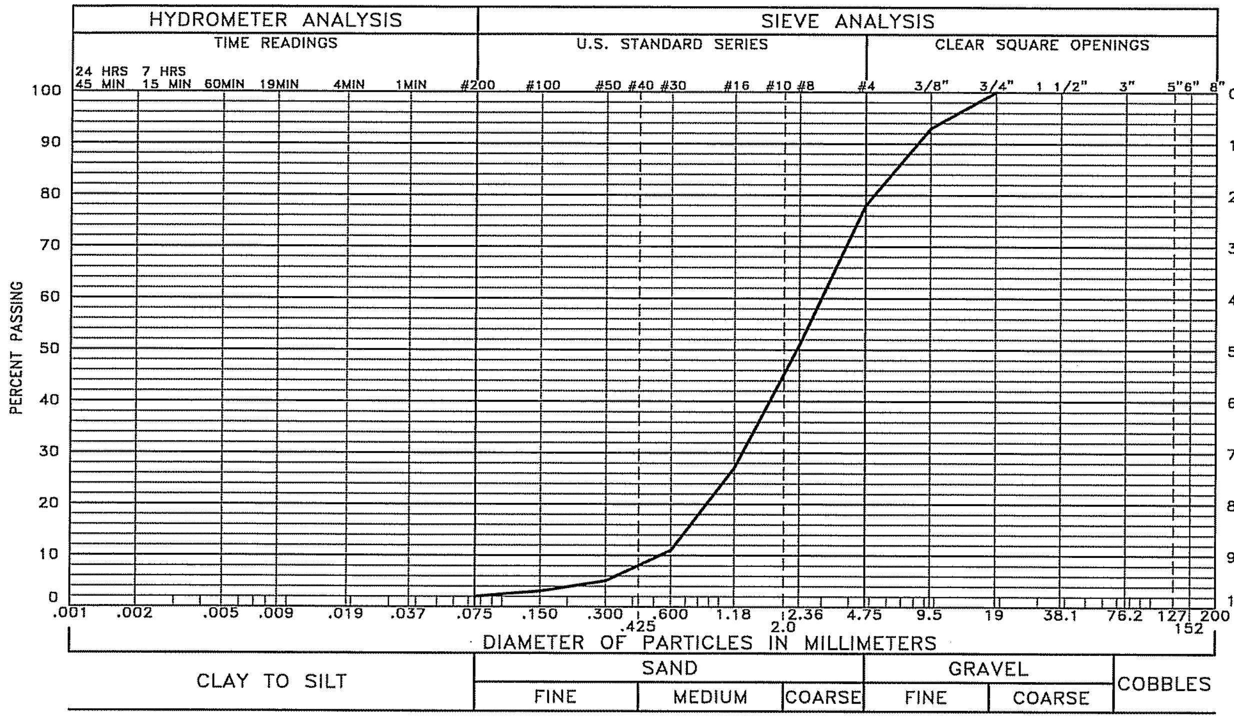
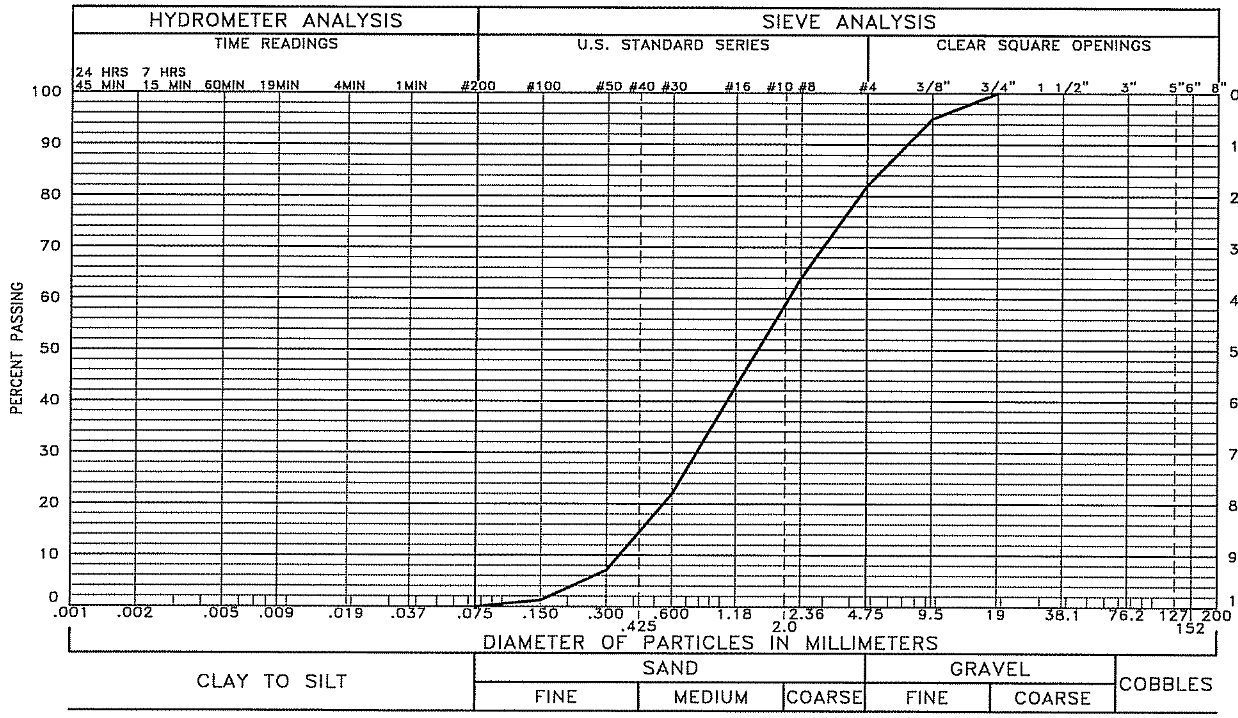


GRAVEL 22 % SAND 76 % SILT AND CLAY 2 %

LIQUID LIMIT PLASTICITY INDEX

SAMPLE OF: Poorly-Graded Gravel with Sand (GP) FROM: Boring SH-1 @ 4'-7'



GRAVEL 18 % SAND 82 % SILT AND CLAY 0 %

LIQUID LIMIT PLASTICITY INDEX

SAMPLE OF: Poorly-Graded Sand with Gravel (SP) FROM: Boring SH-2 @ 0-3'

These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. Sieve analysis testing is performed in accordance with ASTM D422, ASTM C136 and/or ASTM D1140.

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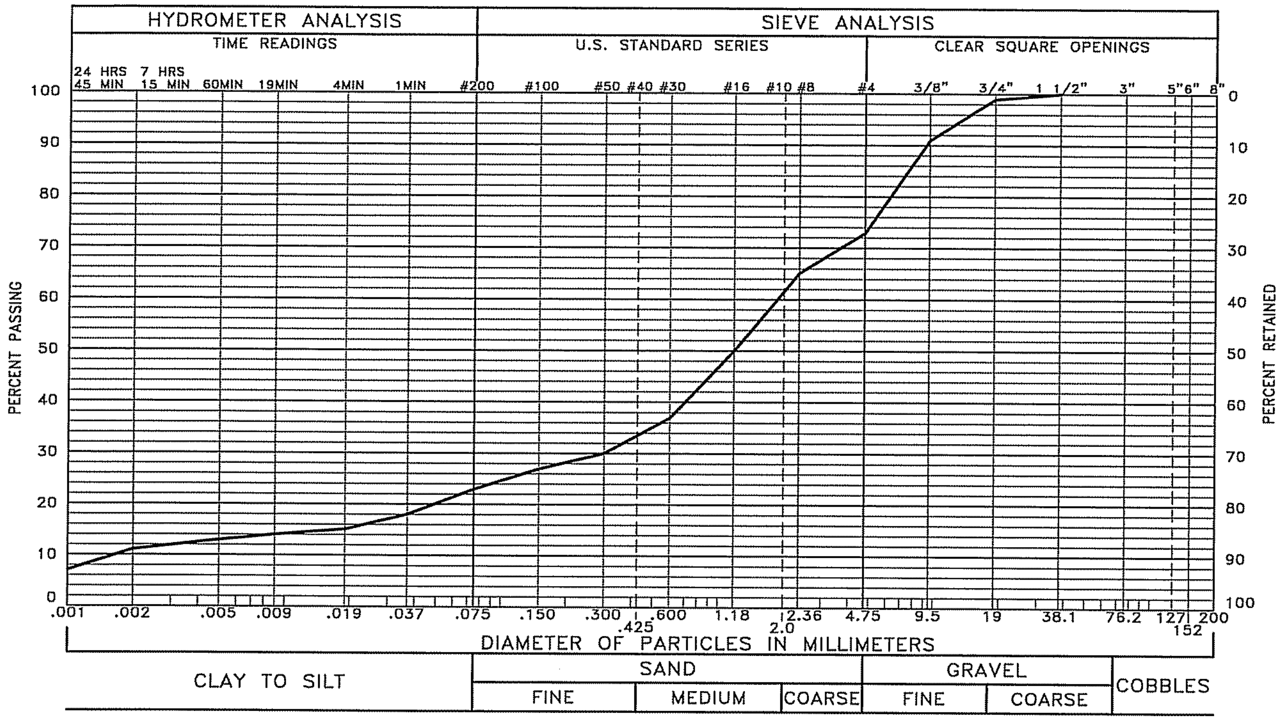
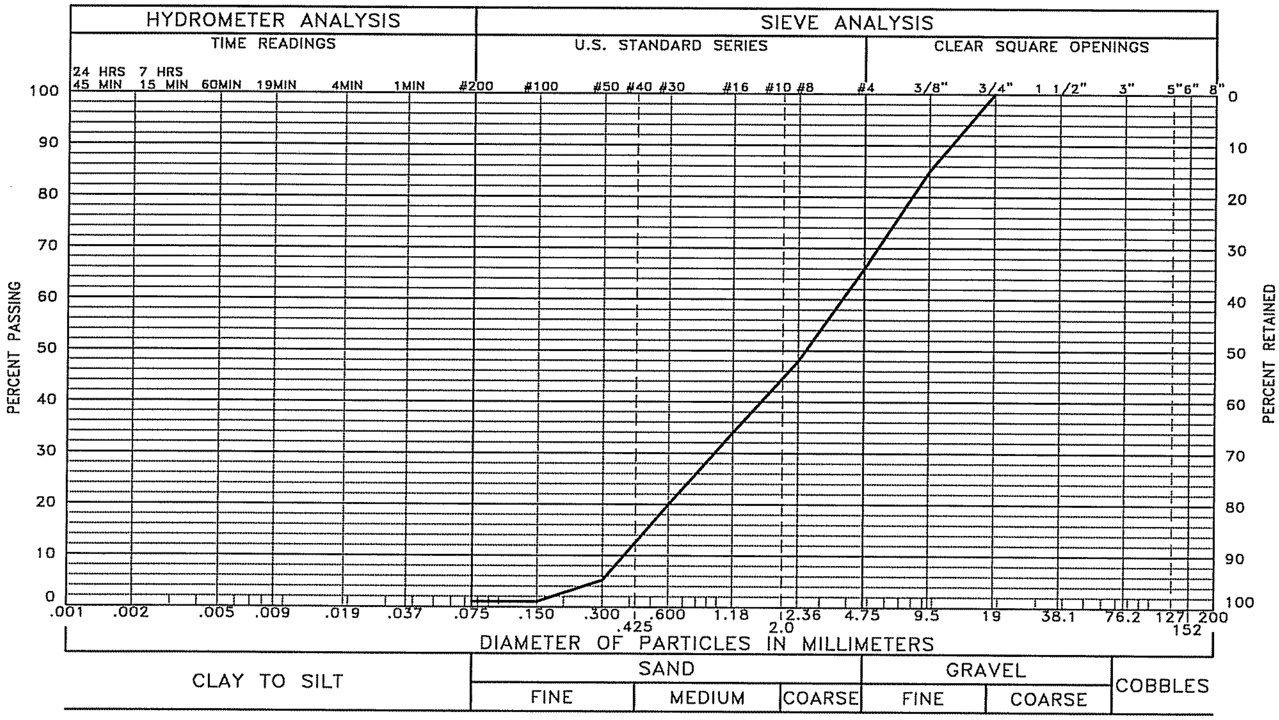
12-1-339	Kumar & Associates	GRADATION TEST RESULTS	Fig. 7



GRAVEL 27 % SAND 50 % SILT AND CLAY 23 %

LIQUID LIMIT PLASTICITY INDEX

SAMPLE OF: Silty Clayey Sand with Gravel (SM-SC) FROM: Boring SH-2 @ 3'-4'



GRAVEL 34 % SAND 65 % SILT AND CLAY 1 %

LIQUID LIMIT PLASTICITY INDEX

SAMPLE OF: Poorly-Graded Sand with Gravel (SP) FROM: Boring SH-3 @ 0-1'

 Sieve analysis testing is performed in accordance with ASTM D422, ASTM C136 and/or ASTM D1140.

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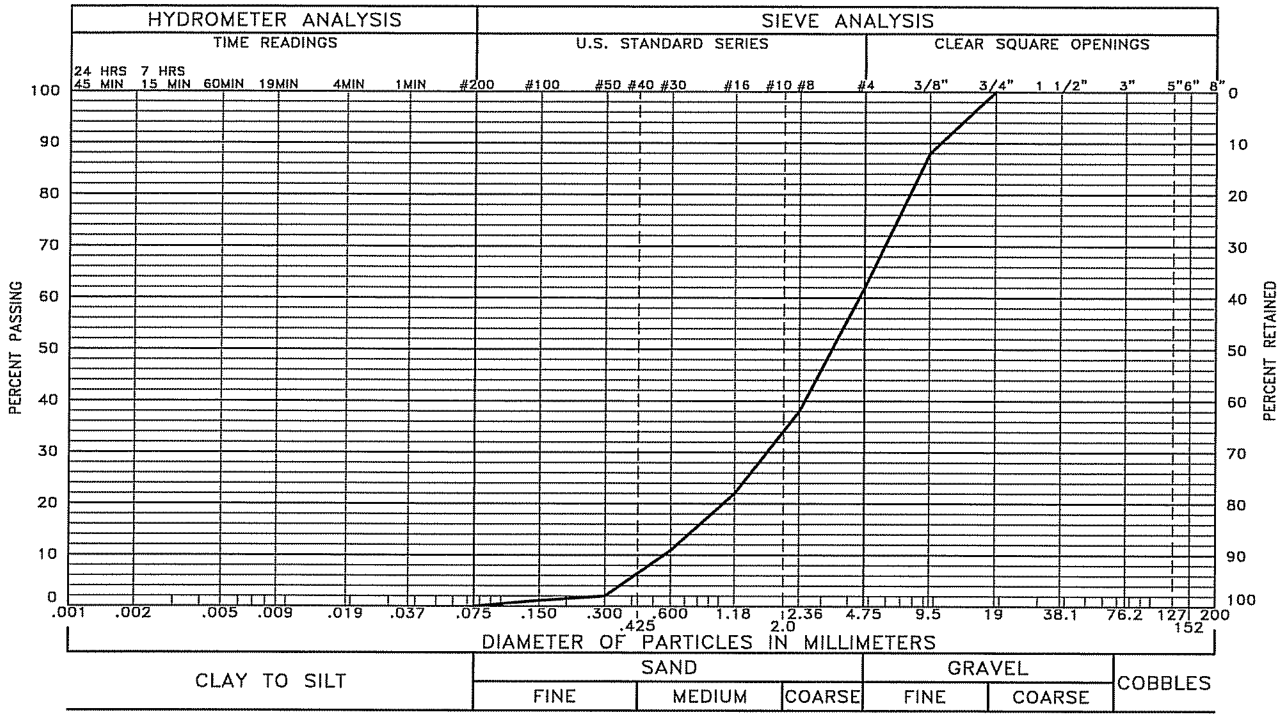
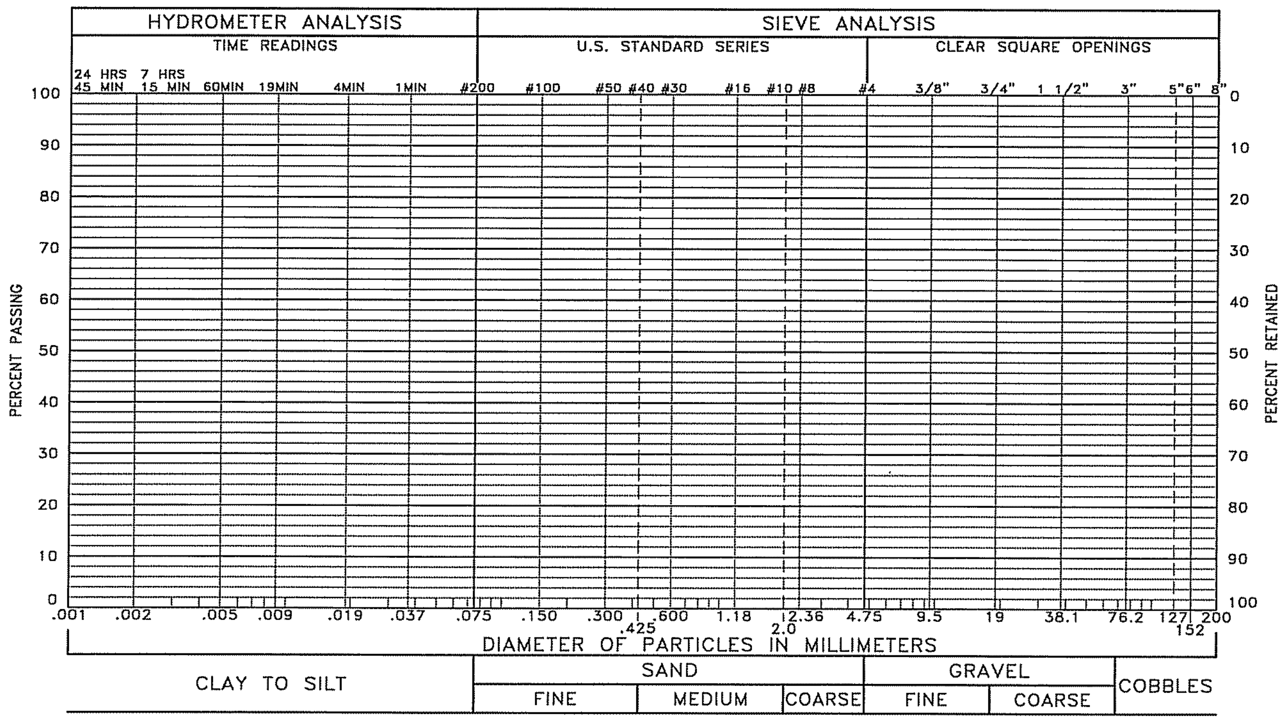
12-1-339	Kumar & Associates	GRADATION TEST RESULTS	Fig. 8



GRAVEL 38 % SAND 62 % SILT AND CLAY 0 %

LIQUID LIMIT PLASTICITY INDEX

SAMPLE OF: Well-Graded Sand with Gravel (SW) FROM: Boring SH-3 @ 2'-3'



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT PLASTICITY INDEX

SAMPLE OF: FROM: Boring

These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. Sieve analysis testing is performed in accordance with ASTM D422, ASTM C136 and/or ASTM D1140.

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12-1-339	Kumar & Associates	GRADATION TEST RESULTS	Fig. 9

TABLE I
SUMMARY OF LABORATORY TEST RESULTS

PROJECT NO.: 12-1-339
PROJECT NAME: South Platte River Recreation and Habitat Improvements
DATE SAMPLED: 7-20-12 and 7-23-12
DATE RECEIVED: 7-25-12

SAMPLE LOCATION		DATE TESTED	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING No. 200 SIEVE	ATTERBERG LIMITS		SOIL OR BEDROCK TYPE
BORING	DEPTH (feet)				GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)	
B-1	14	7-25-12	23.9	93.3			64	47	15	Claystone Bedrock
B-1	19	7-25-12	19.3	101.9						Claystone Bedrock
B-2	1	7-25-12	2.3				4			Poorly-Graded Sand (SP)
B-2	9	7-25-12					6			Poorly-Graded Sand with Silt (SP-SM)
B-2	14	7-25-12					4			Poorly-Graded Sand (SP)
B-3	4	7-25-12	3.3				25			Silty Sand (SM)
B-3	9	7-25-12	21.8	97.5			84	45	18	Claystone Bedrock
B-3	14	7-25-12	16.4	108.2						Claystone Bedrock
B-3	19	7-25-12	19.1	102.1						Claystone Bedrock
B-4	4	7-25-12	38.6	75.4			35	44	15	Fill: Clayey Sand (SC)
B-4	9	7-25-12	17.1	106.1			57	NV	NP	Siltstone Bedrock
B-4	14	7-25-12	16.6	106.7						Claystone Bedrock
B-4	19	7-25-12	21.8	94.0			97	46	24	Claystone Bedrock
SH-1	0-3	7-25-12			22	78	0			Poorly-Graded Sand with Gravel (SP)
SH-1	3-4	7-25-12			31	59	10			Well-Graded Sand with Silty Clay and Gravel (SW-SC)
SH-1	4-7	7-25-12			22	76	2			Poorly-Graded Gravel with Sand (GP)
SH-2	0-3	7-25-12			18	82	0			Poorly-Graded Sand with Gravel (SP)
SH-2	3-4	7-25-12			27	50	23			Silty Clayey Sand with Gravel (SM-SC)
SH-3	0-1	7-25-12			34	65	1			Poorly-Graded Sand with Gravel (SP)
SH-3	2-3	7-25-12			38	62	0			Well-Graded Sand with Gravel (SW)

APPENDIX F

Preliminary (30%) Design Drawings

APPENDIX G

Station Conversion Information

Station Conversion

Design stationing used for the preliminary design follows the designed low flow channel (LFC) alignment that starts at Santa Fe Drive ; whereas, the regulatory stationing is based on the center of the South Platte River (SPR) starting at the Adams and Weld County Line. Due to the continuing increase in curvature between the SPR centerline and the LFC centerline, the conversion equation varies. Figure 1 represents the LFC station versus Station Conversion. The station conversion is the value used to add to the LFC station (or Design station) to receive the SPR Station (or Regulatory Station). Figure 2 presents the SPR station versus the Station Conversion. Use the station conversion to subtract from the SPR station in order to get the LFC station (or Design station).

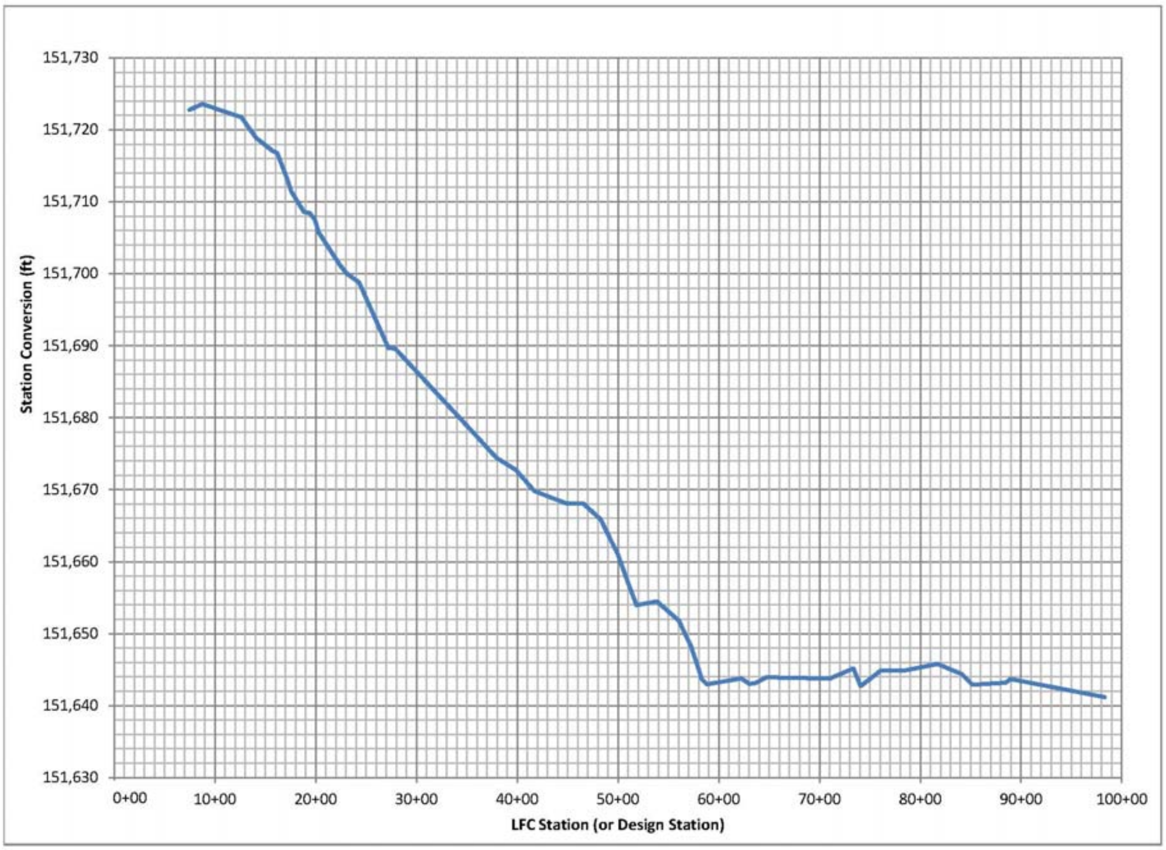


Figure 1. LFC station vs. Station Conversion

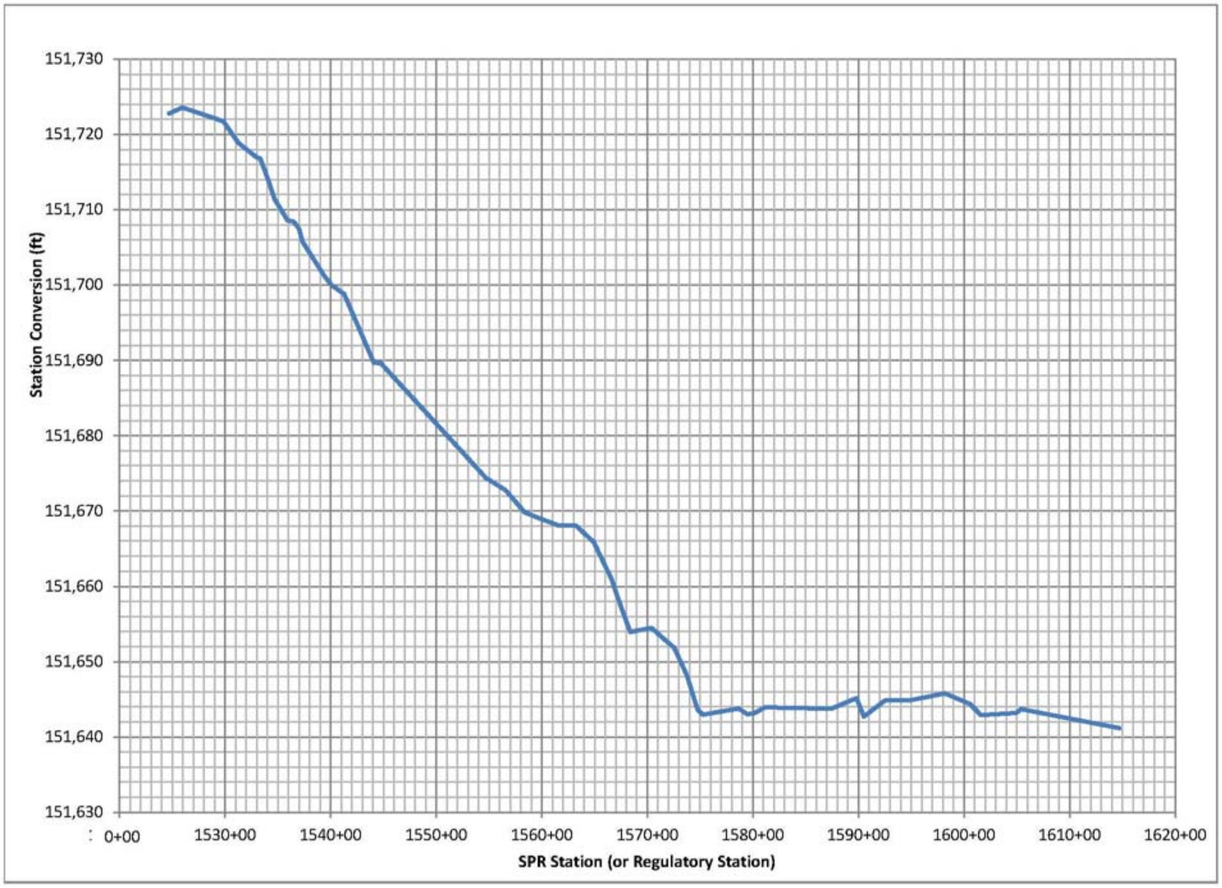


Figure 2. SPR station vs. Station Conversion

APPENDIX H

GOCO Funding Information

Development Project #3 - Grant Frontier/Overland

Proposed Budget

	Source of Funds	Date Secured		GOCO Grant Request	Applicant Match ($)	Partner Match ($)	Amount of CTF Funds ($)	Total Funding ($)
CASH								
	Great Outdoors Colorado	Pending		1,920,909				1,920,909
	City and County of Denver RMNA NRDS Funds	3/14/2012			496,020			496,020
	CWCB WSRA	9/13/2011				250,000		250,000
	Shattuck NRDS Funds	10/13/2011				1,700,000		1,700,000
	UDFCD	2/27/2012				1,242,051		1,242,051
IN-KIND								
	[List Source]							-
	[List Source]							-
	[List Source]							-
TOTAL SOURCE OF FUNDS				**1,920,909**	**496,020**	**3,192,051**	**-**	**5,608,980**

CASH	Use of Funds	Number of Units	Cost Per Unit	GOCO Funds	Applicant Funds	Partner Funds	CTF Funds	Total Funding ($)
CATEGORY 9 - Fish Habitat								
	In-Stream Structural Modifications for River Access	2,000	290	132,000		448,000		580,000
	Modify Existing Drop Structure Near Evans (LS)	1	280,000	228,909		51,091		280,000
	Modify Existing Drop Structure @ Florida (LS)	1	1,200,000	850,000		350,000		1,200,000
CATEGORY 10 - Boat Put-								
	Jetties (Each)	8	47,888	-		383,101		383,101
CATEGORY 11 - Access Trails								
	Access Trails and Trailheads (Each)	4	19,990	79,959				79,959
CATEGORY 12 - Potential								
	Tree Removal/Invasive Species (Each)	550	350	-	17,209	175,192		192,401
	Clear and Grub Vegetative Understory (SF)	400,000	0.15	-	14,963	45,006		59,969
	Promote Growth of Native Vegetation and New	20,000	30	540,087		59,606		599,693
	Bank Stabilization (CY)	3,000	100	-	22,467	277,379		299,846
	Deciduous Tree Planting (Each)	100	500	-	9,969	40,005		49,974
	Shrub/Riparian Plantings (Each)	27,500	6.25	-	24,851	146,936		171,787
	Native/Riparian Seed (SF)	400,000	0.15	-	11,963	48,006		59,969
CATEGORY 13 -								
	Environmental Playground (Each)	1	199,898	-	199,898			199,898
	Environmental Education (Each)	3	29,985	89,954				89,954
Design, Engineering and On the Ground Construction Management		1	852,521		126,120	726,401		852,521
USE OF FUNDS - CASH SUBTOTAL				**1,920,909**	**427,441**	**2,750,723**	**-**	**5,099,072**

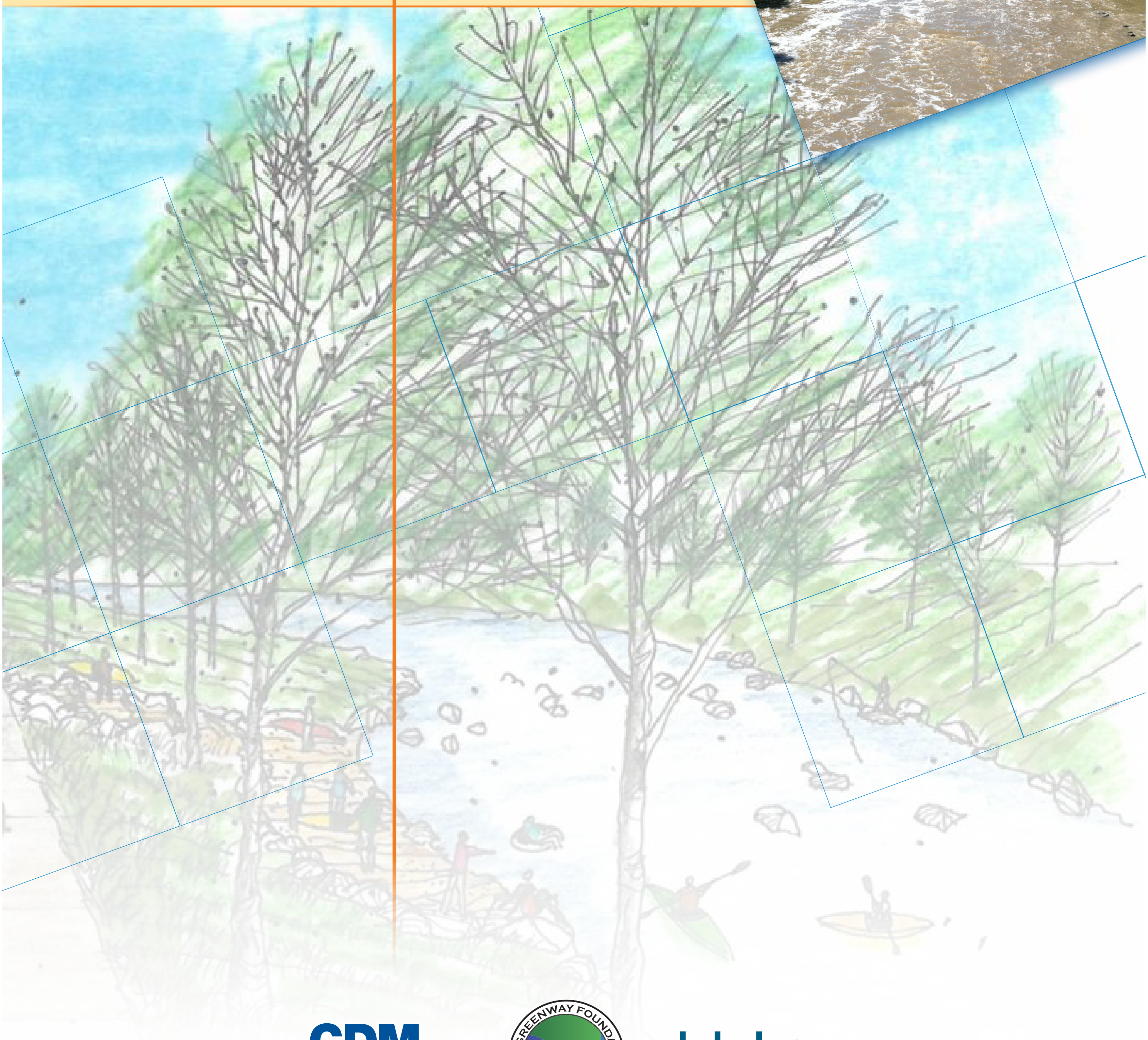
IN-KIND	Use of Funds	No. of Units / Hours	Cost Per Unit / Hour	GOCO Funds	Applicant Funds	Partner Funds	CTF Funds	Total Funding ($)
Professional Services								
vendor/service provider								$0.00
vendor/service provider								$0.00
Materials								
vendor/service provider								$0.00
vendor/service provider								$0.00
Equipment								
vendor/service provider								$0.00
vendor/service provider								$0.00
	USE OF FUNDS - IN-KIND SUBTOTAL				**$0.00**	**$0.00**		**$0.00**
	10% Contingency			**$0**	**$68,579**	**$441,328**	**$0**	**$509,907**
	TOTAL PROJECT COST			**$1,920,909**	**$496,020**	**$3,192,051**	**$0**	**$5,608,980**

CALCULATION OF MATCH REQUIREMENTS

Item	Explanation	Requirement	Actual	Meets Requirement?
Minimum Match	25%/Total Costs	$1,402,245	$3,688,071	Yes
Minimum Cash Match	10%/Total Costs	$560,898	$3,178,163	Yes

CALCULATION OF GOCO %

GOCO % of Total Costs 34.25%



CDM Smith

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