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NAVAJO REGIONAL ROADS

# GEOTECHNICAL EVALUATION

## BRIDGE N651 REPLACEMENT
## TOHATCHI WASH
## NAVAJO ROUTE N108, NEW MEXICO
## BIA PROJECT N108(1)
## WT JOB NO. 2522JW111

Western Technologies Inc.
The Quality People
Since 1955

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Prepared for:

**BIA BRANCH OF ROADS**

September 1, 2002

Registered Professional Engineer (Civil)
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September 1, 2002

BIA – Navajo Regional Office
Branch of Roads, P.O. Box 1060
Gallup, New Mexico 87301

Attn: Mr. Corwyn D. Henry

Re: Geotechnical Evaluation
Bridge N651 Replacement
Tohatchi Wash – Navajo Route N108
McKinley County, New Mexico
BIA Project N108(1)

Job No. 2522JW111

Western Technologies Inc. has completed the geotechnical evaluation for the proposed replacement of the N651 Bridge located over Tohatchi Wash on Navajo Route N108 in McKinley County, New Mexico. This study was performed in general accordance with our proposal number 3121PC041 dated April 26, 2001 and with Contract No. CMN00001100. The results of our study, including the boring location diagram, laboratory test results, boring logs, and the geotechnical recommendations are attached.

We have appreciated being of service to you in the geotechnical engineering phase of this project and are prepared to assist you during the construction phases as well. If design conditions change, or if you have any questions concerning this report or any of our testing, inspection, design and consulting services, please do not hesitate to contact us. We look forward to working with you on future projects.

Sincerely,
WESTERN TECHNOLOGIES, INC.
Geotechnical Engineering Services

Craig P. Wiedeman

Craig P. Wiedeman, P.E.

Copies to: Addressee (3)

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## 1.0 PURPOSE

This report contains the results of our geotechnical evaluation for the proposed bridge to replace an existing bridge over Tohatchi Wash on Navajo Route N108. The site is located in the community of Tohatchi in McKinley County, New Mexico. The purpose of these services is to provide information and recommendations regarding:

- general subsurface conditions including soil, bedrock and groundwater conditions
- foundation design parameters for bridge support
- lateral earth pressures
- earthwork
- drainage adjacent to roadway embankments

Results of the field exploration, field tests, and laboratory testing program are presented in the Appendices.

## 2.0 PROJECT DESCRIPTION

Project information supplied by Mr. Corwyn D. Henry indicates the proposed structure will consist of a single-span, Type V AASHTO prestressed concrete beam bridge with a cast-in-place concrete deck. Preliminary design plans indicate that the proposed bridge will be approximately 34 meters long and approximately 12 meters wide. It is anticipated that the bridge deck surface grade will be located about 6 meters above the existing flow line elevation of the wash. The centerline of the new structure will be located on or very close to the centerline of the existing bridge. The orientation of the new bridge will match the orientation of the existing bridge. The design (100-year) flood flow will be 63.8 cubic meters per second with a velocity of 4.37 meters per second and a water surface elevation of 1953.44 meters. For the purpose of this evaluation, the preliminary scour depth is assumed to be 5 meters. The loads for the proposed structure are as follows:

| | DL | LL – Case 1 | | LL – Case 2 | |
|---|---|---|---|---|---|
| | P | P | $M_T$ | P | $M_T$ |
| Abutments | 3501.5 kN | 789.1 kN | 649.0 kN-m | 584.5 kN | 1548.8 kN-m |

where:

| | |
|---|---|
| DL: | dead load |
| LL: | vehicular live load (HS-20-44/MS-18) |
| P: | vertical reaction for entire substructure unit |
| $M_T$: | moment about transverse axis of structure (strong axis of substructure unit) |
| Case 1: | vehicular loading for maximum vertical reaction |
| Case 2: | vehicular loading for maximum transverse moment |

## 3.0 SCOPE OF SERVICES

### 3.1 Field Exploration

A total of four borings were advanced at the project site. Two borings were drilled to depths of about 19.5 and 28.3 meters in the west abutment area, and two borings were drilled to depths of about 21.7 and 30.6 meters in the east abutment area. The borings were advanced alternately on opposite sides at both of the proposed abutment locations. The borings were drilled at the locations shown on the Boring Location Diagram. The elevations of the top of the borings were established by measurements with an engineer's level relative to existing grade at the approximate centerline of the east end of the existing bridge, using an approximate elevation of 1958.5 meters based on contour information provided by ASCG. All drilling and sampling was performed in general accordance with applicable ASTM or AASHTO Standards.

The borings were advanced with a CME-75 truck mounted drilling rig, utilizing continuous flight, hollow stem auger, ODEX and rotary air drilling methods to the full depth of exploration. Relatively undisturbed samples of the soils encountered at the boring locations were obtained using a ring barrel sampler. Disturbed samples of the soils encountered were obtained using a standard split spoon sampler. Penetration resistance measurements were taken for each ring barrel and split spoon sample by driving the sampler with a 63.5 kilogram hammer, falling 762 millimeters. A sample of the underlying rock material encountered in Boring 2 was obtained using an NX core barrel. Rock Quality Designation (RQD) and percent recovery were determined for the core run. Three samples of the near surface channel bed soils were obtained for gradation analyses for use in sediment transport analysis to be performed by others. All samples retrieved during the field exploration were sealed to preserve moisture and returned to our laboratory for further evaluation and testing. Boring logs are presented in Appendix A.

A field log was prepared for each boring. These logs contain visual classifications of the materials encountered during drilling as well as interpolation of the subsurface conditions between samples. Final logs, included in Appendix A, represent our interpretation of the field logs and include modifications based on laboratory observations and tests of the field samples. The final logs describe the materials encountered, their thicknesses, and the locations where samples were obtained. The Unified Soil Classification System was used to classify soils. The soil classification symbols appear on the boring logs and are briefly described in Appendix A.

### 3.2 Laboratory Analyses

Laboratory analyses were performed on representative soil samples to aid in material classification and to estimate pertinent engineering properties of the on-site soils for preparation of this report. Testing was performed in general accordance with applicable ASTM specifications. The following tests were performed and the results are presented in Appendix B.

- Direct shear
- Gradation
- Plasticity
- pH
- Resistivity
- Soluble salts and sulfates
- Maximum - minimum density

### 3.3 Analyses and Report

This geotechnical evaluation report includes a description of the project, a discussion of the field and laboratory testing programs, a discussion of the subsurface conditions, and design recommendations as required to satisfy the purpose previously described. This report is for the exclusive purpose of providing geotechnical engineering and/or testing information and recommendations. The scope of services for this project does not include, either specifically or by implication, 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. We are available to discuss the scope of such studies with you.

## 4.0 SITE CONDITIONS

### 4.1 Surface

At the time of the field exploration, the project site consisted of the existing crossing of Bridge N651. The existing bridge consists of a 32.65 meter long, single span, steel frame structure supported by a combination of both concrete and wood abutments. The bridge structure and the metal decking appear to be in fair condition. This area of Tohatchi Wash consists of a wide and shallow crossing. The slopes on both sides of the existing bridge abutment appear to be fills that were placed in order to elevate the bridge above the lower channel banks along this portion of the wash. The existing bridge deck is approximately 6 meters above the bottom of the wash. Many of the channel banks along the wash have near vertical slopes, which appear to have eroded and widened the wash on both the upstream and downstream sides. The surface of the channel banks and the wash bottom are covered with cobbles and boulders ranging from about 150 millimeters to 1.8 meters in nominal dimension. Vegetation in the immediate area of the crossing consists of a sparse growth of cottonwood trees, elm trees, and other native shrubs and grass.

### 4.2 Subsurface

As presented on the Logs of Borings, surface and subsoils to depths of about 0.9 to 7.5 meters were found to be loose to medium dense silty and clayey sands with a trace to some gravel and cobbles. The surface soils to a depth of about 2.9 meters in Boring 4 appeared to be existing fill material. The sands were underlain by medium dense to dense gravel and cobbles extending to depths of about 8.2 to 11.6 meters. The gravel and cobbles were underlain by soft to moderately hard clayshales that extended to the full depth of exploration in all of the borings. The logs in Appendix A show details of the subsurface conditions encountered during the field exploration.

### 4.3 Groundwater

During the field exploration, groundwater was encountered in all of the borings at depths of 5.2 to 8.5 meters below existing site grades. These observations represent the conditions at the time of exploration and may not be indicative of other times. Groundwater levels and soil moisture conditions can be expected to fluctuate with varying local and regional seasonal and perennial weather conditions, changes in surface and subsurface drainage patterns, flow conditions in the wash, adjacent construction or development, and other factors.

### 4.4 Geology

The Menefee Formation of late Cretaceous age forms the surface for most of the area surrounding Tohatchi. The dominant feature in the area is the Chuska Mountains. High on the southeast end of the mountains are the Densa Bluffs, cliffs of Chuska Sandstone. This sandstone formation is of uncertain age. Because there are dikes and volcanic deposits of Oligocene and Pliocene age cutting through and overlaying the Chuska, it is assumed that the sandstone was deposited earlier in the Tertiary period. The lower Chuska consists of stream deposits, while the upper portion is of aeolian origin.

## 5.0 GEOTECHNICAL PROPERTIES AND ANALYSIS

### 5.1 Laboratory Tests

The results of the direct shear testing on the near surface soils indicated cohesion (c) and friction angle ($\phi$) values of 9.6 to 20.3 kPa and 31 to 35 degrees, respectively. The results of the direct shear testing on the underlying clayshale material indicated cohesion (c) and friction angle ($\phi$) values of 43.1 kPa and 11 degrees, respectively. The values of 9.6 kPa and 31 degrees were used in the analysis. The pH of the subsurface soils measured as 7.53, and the minimum resistivity value measured as 2168 ohm-cms. The soluble sulfates concentrations in the samples tested ranged from 10 to 295 ppm.

### 5.2 Field Tests

Existing subsoils exhibited low to moderate resistance to penetration using the standard penetration test method (ASTM D1586) and test method ASTM D3550. The penetration resistances also exhibited some variability between test locations. The penetration resistance can be a useful index of the consistency or relative density of the materials encountered.

## 6.0 RECOMMENDATIONS

### 6.1 General

Recommendations contained in this report are based on our understanding of the project criteria described in Section 2.0, Project Description, and the assumption that the soil and subsurface conditions are those disclosed by the borings. Others may change the plans, final elevations, number and type of structures, foundation loads, and other factors during design or construction. Substantially different subsurface conditions from those described herein may be encountered or become known. Any changes in the project criteria or subsurface conditions shall be brought to our attention in writing.

### 6.2 Foundations

Due to the medium dense to dense gravel and cobble layer encountered at depths of about 0.9 to 7.5 meters below existing site grades in all of the borings, construction of deep foundations systems such as driven H-piles or drilled shafts will be difficult on this site. Accordingly, recommendations are provided for supporting the bridge abutment loads on continuous, shallow spread footings bearing on engineered fill material. As requested, recommendations for deep foundation systems are also provided.

Our services did not include a cost study. Therefore, we do not know which of the possible foundations alternatives would be most economical. We recommend that the client and his design consultant team evaluated the design criteria presented herein, discuss the various alternatives with a knowledgeable foundation contractor and then prepare a detailed cost study or receive bid alternates to determine which system would be the most appropriate.

### 6.2.1 Shallow Foundations

Bearing on shallow spread footings assumes that adequate scour countermeasures are implemented to prevent scour from occurring below all foundation elements. Unprotected foundations should be designed to bear a minimum of 0.9 meters below potential scour depth anticipated near the channel walls. Construction of cutoff walls may be required if potential scour can not be mitigated in these areas. Alternative footing depths (measured below the lowest adjacent finished grade), widths, design bearing capacities, and estimated settlements are presented in the following tabulation:

| Footing Depth (m) | Footing Width (m) | $q_{all}$ (kPa) | $S_e$ (cm) |
|---|---|---|---|
| 1.0 | 1.0 | 297.96 | 2.12 |
| | 1.5 | 335.38 | 3.06 |
| | 2.0 | 372.80 | 4.14 |
| | 2.5 | 410.23 | 5.25 |
| 1.5 | 1.0 | 357.37 | 2.55 |
| | 1.5 | 394.79 | 3.60 |
| | 2.0 | 432.22 | 4.80 |
| | 2.5 | 469.64 | 6.02 |
| 2.0 | 1.0 | 416.78 | 2.97 |
| | 1.5 | 454.20 | 4.14 |
| | 2.0 | 491.63 | 5.46 |
| | 2.5 | 529.05 | 6.78 |

The following criteria should be used in determining the minimum extent of engineered fill required below and beyond the edges of the shallow footings:

| | |
|---|---|
| Minimum depth of recompaction/engineered fill (below the bottom of the footing) | Equal to the footing width (not less than 0.6 meters) |
| Minimum width of recompaction/engineered fill (beyond the footing edges) | Equal to 0.5 times the depth of recompaction below footings (not less than 0.6 meters) |

After any overexcavation has been accomplished, the exposed soils should be scarified, moistened or dried as required, and compacted to a minimum depth of 20 centimeters. This depth may be included in the minimum required depth of compaction below footings.

For foundations adjacent to slopes, a minimum horizontal setback of 1.5 meters should be maintained between the foundation base and slope face. In addition, the setback should be such that an imaginary line extending downward at 45 degrees from the nearest foundation edge does not intersect the slope.

We recommend that the geotechnical engineer or his representative observe the foundation excavations before reinforcing steel and concrete are placed. It should be determined whether the soils exposed are similar to those anticipated for support of the foundations. Any soft, loose or unacceptable soils should be undercut to suitable materials and backfilled with approved fill materials or lean concrete.

### 6.2.2 Driven Piles

Driven steel piles designed in end bearing may be used for support of the bridge abutments. The design capacity of a single driven pile is a function of several factors including: 1) the size and type of pile, 2) the dynamics and efficiency of the pile hammer, and 3) the engineering properties of the subsurface materials. The most effective means of determining pile capacities for either tensile or axial loads is through pile load tests. Alternately, dynamic driving formulas can be used to assess pile capacity of production piles.

Driven piles may be supported on the moderately hard to hard shale encountered at depths of about 8 to 11.5 meters below existing site grades. Due to the medium dense to dense gravel and cobble layer encountered at depths of about 0.9 to 7.5

meters below existing site grades in all of the borings, predrilling of the pile locations will probably be required to penetrate the gravel and cobble layer. Preliminary design criteria have been formulated and are presented below.

We anticipate that steel H-piles will be the preferred pile type utilized. Based upon the field exploration and laboratory testing program, axial capacities for four sizes of steel H-piles are presented here. Allowable capacities were based on 25 percent of the allowable steel stress. Capacities of other pile types or sizes can be provided, if requested. The design capacity is for dead plus live loads. A one-third increase may be used when considering loads including wind or seismic forces.

| Pile Section | Allowable Axial Capacity (kN) |
|---|---|
| HP 10x42 | 496.4 |
| HP 12x53 | 620.5 |
| HP 12x74 | 872.7 |
| HP 14x102 | 1201.0 |

Final allowable capacities of pile foundations should be determined in the field during construction, using an appropriate dynamic driving formula and/or testing apparatus. A driving hammer compatible with the size and type of the specified piles should be used. We recommend a hammer delivering a minimum rated energy of 20,337 joules. All pile driving equipment should be approved before use. The group capacity of piles supported on shale is the number of piles times the individual capacity of each pile. The use of reinforced heavy-duty pile tips is recommended to properly embed the piles into the shale.

### 6.2.3 Drilled Shafts

Drilled shafts socketed into shale may also be used to support the bridge structure. The allowable axial capacities of various diameters and rock socket lengths of drilled shafts are presented in Plate 3. A factor of safety of 2.5 was used in the calculations. Capacities of other diameters and rock socket lengths can be provided, if requested. A minimum rock socket length of 1.5 meters should be used. The design capacities shown are for dead plus live loads and includes the weight of the shaft. A one-third increase may be used when considering wind or seismic loads.

Estimated settlements for drilled shafts are 25 millimeters or less for maximum anticipated loads. Shafts in groups should be spaced at least three diameters on centers. There will be no reduction in the downward capacities of the shafts due to group action if the shafts are spaced as recommended.

Shafts in groups should be drilled and filled alternately, allowing the concrete to set at least eight hours before drilling an adjacent shaft. Shaft excavations should not be allowed to stand open overnight, concrete should be placed as soon as possible after inspection. Protective steel casing or a drilling slurry may be required to hold the excavation open, particularly when groundwater is encountered. The casing should be removed as the concrete is placed. However, the protective steel casing should not be removed until the concrete is above the groundwater level and any caving soils. A minimum head of 3 meters of concrete should be maintained above the bottom of the casing during withdrawal, and the contractor should prevent concrete from "hanging-up" inside the shell which can cause soil and water intrusion below the casing. We recommend that the concrete be placed into the drilled shaft through a tremie pipe. The concrete should be maintained above the bottom of the tremie pipe, so that any water will be forced out the top of the hole during concrete placement. Drilling to design depths will require the use of heavy-duty rigs, probably combined with coring techniques, to penetrate the medium dense to dense gravel and cobble layer encountered at depths of about 0.9 to 7.5 meters below existing site grades in all of the borings.

### 6.3 Lateral Design Criteria

Lateral load criteria was developed from data outlined in Documentation of Computer Program LPILE1, 1985, by Ensoft, Inc. To resist forces in the horizontal direction, piles or drilled shafts should be designed for lateral loads using the following parameters:

| Material | Coefficient of Horizontal Subgrade Reaction (kPa/m) | Unit Weight ($kg/m^3$) |
|---|---|---|
| granular soils | 27,150 | 1810 |
| shale | 245,000 | 2000 |

Due to potential scour effects, resistance to lateral loads should be neglected within the calculated scour zone. Once the final scour analysis has been performed and more detailed information regarding the type of foundation selected becomes available, a lateral analysis should be conducted. We are able to assist you in this matter, if requested.

The lateral pressures appropriate for design of the abutments and appurtenant structures will be a function of the type of material used as backfill, the type of undisturbed soils, and the magnitude of lateral movement permitted to occur in the abutments and appurtenant structures. For cantilevered walls above any free water surface with level backfill and no surcharge loads, recommended equivalent fluid pressures and coefficients of base friction for unrestrained elements are:

- Active:
  Undisturbed subsoil .................... 5.7 kPa/m
  Compacted granular backfill .................... 4.7 kPa/m
  Compacted site soils .................... 5.5 kPa/m

- Passive:
  Undisturbed subsoil .................... 39.3 kPa/m*

  * Passive pressure resistance should be neglected over the maximum depth of anticipated scour.

- Coefficient of base friction:
  acting alone .................... 0.40
  acting in conjunction with passive pressure .................... 0.30

Where the design includes restrained elements, the following equivalent fluid pressures are recommended:

- At-rest:
  Undisturbed subsoil .................... 10.2 kPa/m
  Compacted granular backfill .................... 8.6 kPa/m

The equivalent fluid pressures presented herein do not include the lateral pressures arising from the presence of:

- hydrostatic conditions due to submergence or partial submergence
- positively or negatively sloping backfill
- permanent or temporary point surcharge loading
  seismic or dynamic conditions

We should be contacted for additional recommendations if any of these conditions will exist.

In regard to the abutments, wheel loads transmitted to the backfill immediately behind the walls will result in increasing the lateral earth pressures. An allowance for this increase in earth pressure should be included in the analysis and design of the abutments. This can be accomplished by assuming that the wheel loads are equivalent to a uniformly distributed loads. The uniform surcharge is then converted to an additional thickness of backfill by dividing this value by the unit weight of the backfill. The unit weight of the undisturbed soils can be estimated as 1762 $kg/m^3$. The unit weight of compacted granular backfill can be estimated to be 2000 $kg/m^3$.

If heavily reinforced portland cement concrete approach slabs are constructed on and adjacent to the abutments, the allowance for additional lateral earth pressure caused by wheel loads may be neglected. These slabs should extend a minimum of 4.6 meters beyond the abutments. The principal purpose of such construction is to provide a slab with sufficient strength to transfer wheel loads to the abutments.

We recommend a free-draining soil layer or manufactured geosynthetic material be constructed adjacent to the back of the wall. A filter may be required between the soil and backfill and drainage layer. This drainage zone should help prevent development of hydrostatic water pressure on the wall. The vertical drainage zone should be tied into a gravity drainage system at the base of the wall. In lieu of a drainage layer, weep holes should be provided for draining any water that could accumulate in the backfill or native soils. Weep holes should have a maximum spacing of 3.1 meters. It is important that all backfill be properly placed and compacted. Backfill should be mechanically compacted in layers. Flooding or jetting should not be permitted. Care should be taken not to damage the walls when placing the backfill. Backfill should be observed and tested during placement.

Fill against footings, abutment walls, and wing walls should be compacted to densities specified in **Earthwork.** Compaction of each lift adjacent to walls should be accomplished with hand-operated tampers or other lightweight compactors. Overcompaction may cause excessive lateral earth pressures which could result in wall movements.

## 6.4 Seismic Considerations

The following seismic design criteria are based upon the 1996 AASHTO *Standard Specifications for Highway Bridges*. Based upon the nature of the subsurface materials and the horizontal acceleration map included in Section 3, we recommend using an effective peak acceleration coefficient (A) of 0.02. In addition, we recommend using a site coefficient (S) of 1.0 and a Seismic Performance Category (SPC) of A.

### 6.5 Drainage

In order to maintain stability of the roadway and the approaches adjacent to the bridge structure, positive drainage should be provided during construction and maintained throughout the life of the proposed roadway. Water should not be allowed to pond at the base of the embankments.

### 6.6 Corrosivity

We recommend a Type II portland cement be used for all concrete on and below grade.

A minimum resistivity value of 2168 ohm-cms and a pH value of 7.53 was determined from laboratory testing of the on-site soils. These results indicate that the on-site soils are considered to exhibit low to moderate corrosive potential to steel underground piping. The information derived from the testing should be used as an aid in choosing the construction materials that will be in contact with these soils and that will need to be resistant to various corrosive forces. Manufacturer's representatives should be contacted regarding the specific corrosivity resistance for their particular product.

## 7.0 EARTHWORK

### 7.1 General

The conclusions contained in this report for the proposed construction are contingent upon compliance with recommendations presented in this section. Any excavating, trenching, or disturbance which occurs after completion of the earthwork must be backfilled, compacted and tested in accordance with the recommendations contained herein. It is not reasonable to rely upon our conclusions and recommendations if any future unobserved and untested trenching, grading or backfilling occurs.

### 7.2 Clearing and Grubbing

Clearing and grubbing should be performed in accordance with Section 201 of the *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects*, FP-96, referred to hereinafter as the *Standard Specifications*.

### 7.3 Removal of Structures and Obstructions

Removal of all existing structures and obstructions should be performed in accordance with Section 203 of the *Standard Specifications*.

## 7.4 Excavation, Embankment, and Foundation Preparation

Excavation, bridge approach embankments and foundation preparation should be performed in accordance with Section 204 of the *Standard Specifications*.

## 7.5 Structure Excavation and Backfill

Structure excavation and backfill should be performed in accordance with Section 208 of the *Standard Specifications*.

## 7.6 Structural Concrete

Structural concrete should conform to Section 552 of the *Standard Specifications*.

## 7.7 Driven Piles

Installation of driven piles should be performed in accordance with Section 551 of the *Standard Specifications*.

## 7.8 Drilled Shaft Construction

Drilled shafts should be constructed in accordance with Section 565 of the *Standard Specifications*.

## 7.9 Structural Backfill

Structural backfill should conform to Section 704.04 of the *Standard Specifications*.

## 7.10 Piling

Driven piling materials should conform to Section 715 of the *Standard Specifications*.

## 7.11 Scour Countermeasures

Rock for scour countermeasures should conform to Section 705 of the *Standard Specifications*.

## 8.0 LIMITATIONS

This report has been prepared based on our understanding of the project criteria as described in Section 2.0. Others may make changes in the project criteria during design or construction, and substantially different subsurface conditions may be encountered or become known. The conclusions and recommendations presented herein shall not continue to be valid unless all variations are brought to our attention in writing, and we have had an opportunity to assess the effect such variations may have on our conclusions and recommendations and respond in writing.

The recommendations presented are based upon data derived from a limited number of samples obtained from widely spaced borings. The attached logs are indicators of subsurface conditions only at the specific locations and times noted. The geotechnical engineer necessarily makes assumptions as to the uniformity of the geology and soil structure between borings, but variations can exist. Accordingly, whenever any deviation or change is encountered or become known during design or construction, the conclusions and recommendations presented herein shall not continue to be valid unless WT is notified in writing, has actually reviewed the matter, and has issued a written response.

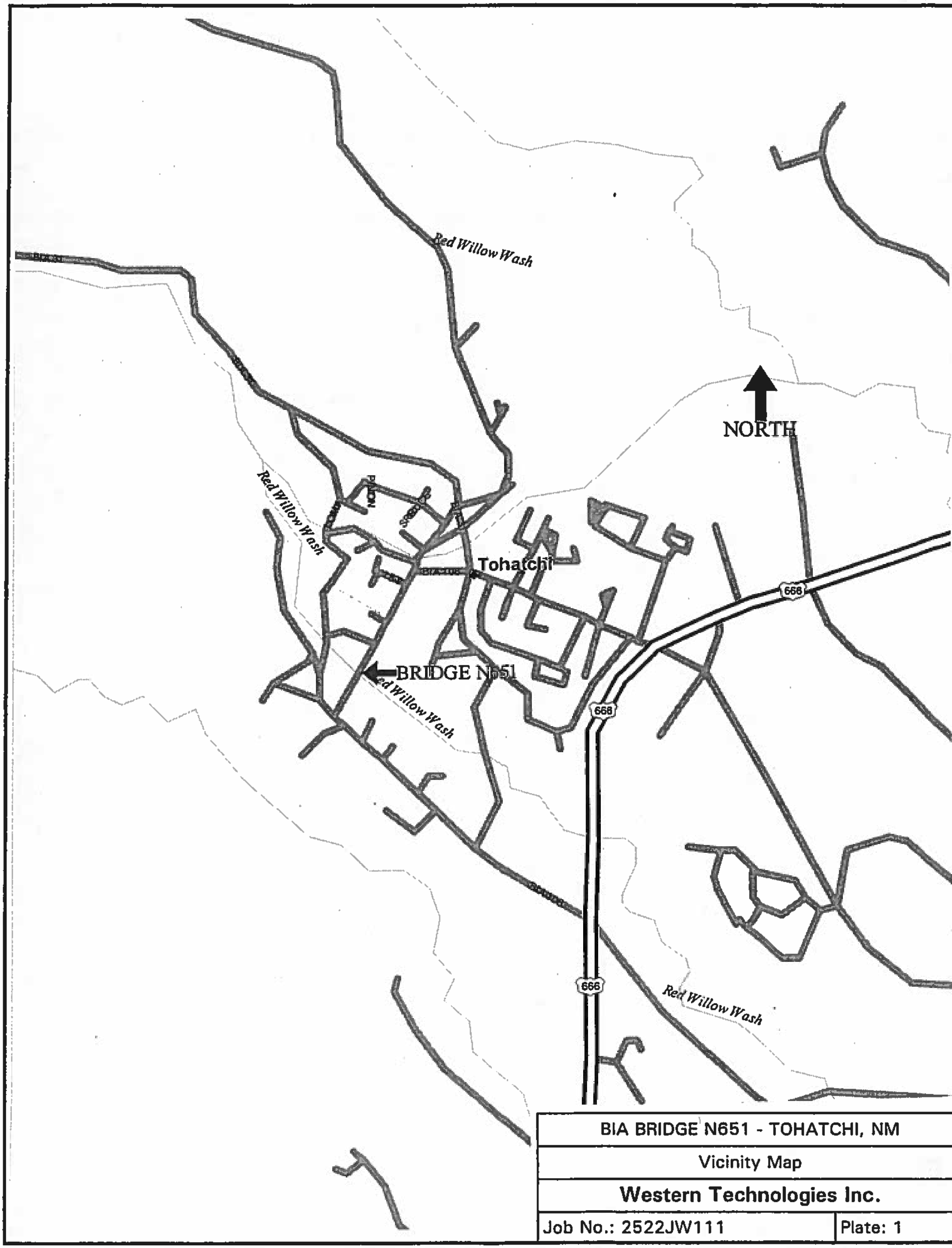
This report does not provide information relative to construction methods or sequences. Any person reviewing this report must draw his own conclusions regarding site conditions as they relate to the employment or development of construction techniques. This report is valid for one year after the date of issuance unless there is a change in circumstances or discovered variations justifying an earlier expiration of validity. After expiration, no person or entity has any right to rely on this report without further review and reporting by WT under a separate contract.

## 9.0 OTHER SERVICES

The geotechnical engineer should be retained for a general review of final plans and specifications to determine compliance with our recommendations. The geotechnical engineer should also be retained to provide observation and testing services during excavation, earthwork operations, and foundation and construction phases of the project. Observation of footing excavations should be performed prior to placement of reinforcing and concrete to confirm that satisfactory bearing materials are present.

## 10.0 CLOSURE

We have prepared this report as an aid to the designers of the proposed project. The comments, statements, recommendations, and conclusions set forth in this report reflect the opinions of the authors. These opinions are based upon conditions at the location of specific tests and observations, and on the data developed to satisfy the scope of services defined by the contract documents. Work on your project was performed in accordance with generally accepted industry standards and practices by professionals providing similar services in this locality. No other warranty, express or implied, is made.

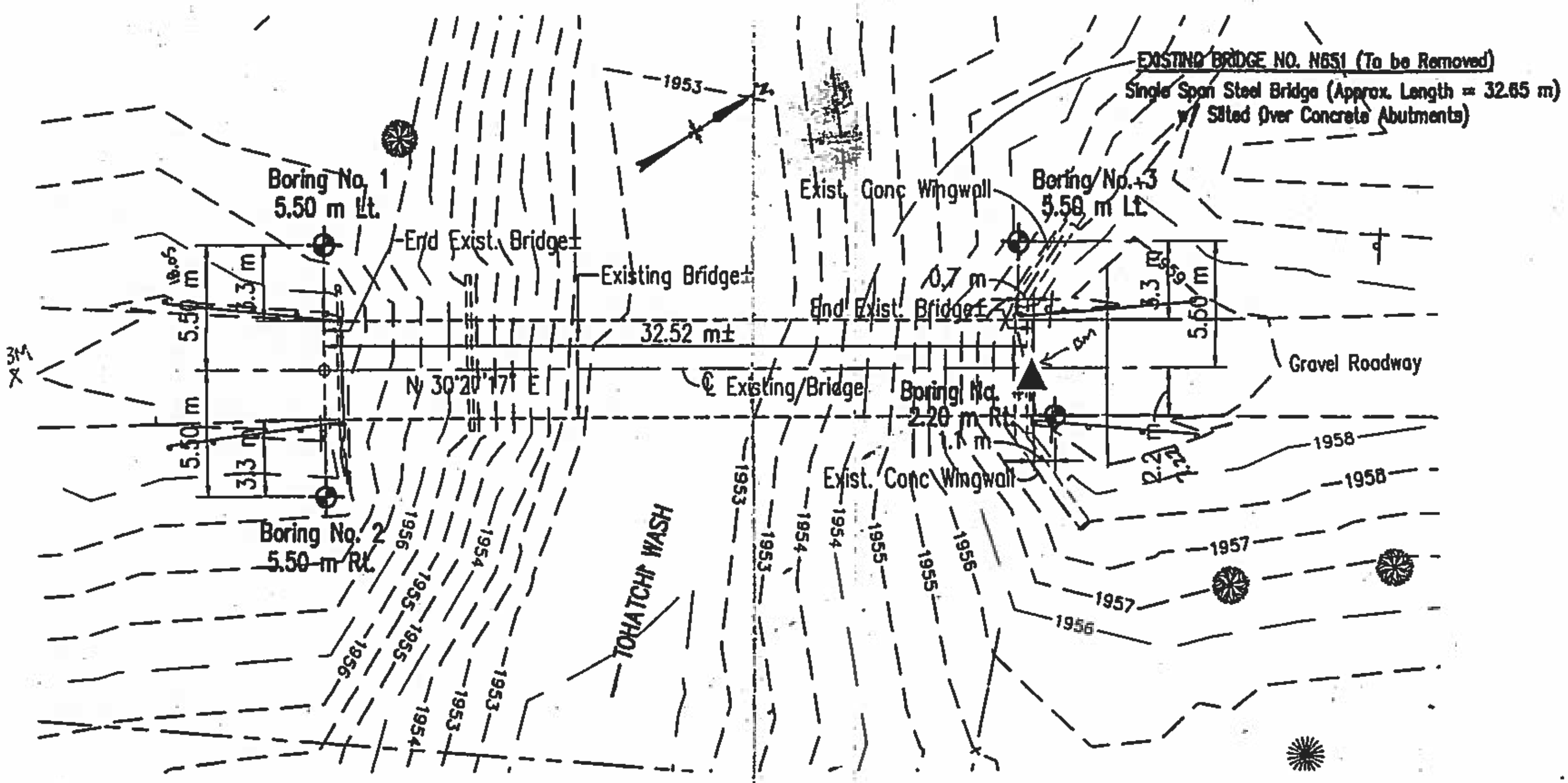



BIA BRIDGE N651 - TOHATCHI, NM

Vicinity Map

**Western Technologies Inc.**

| | |
|---|---|
| Job No.: 2522JW111 | Plate: 1 |

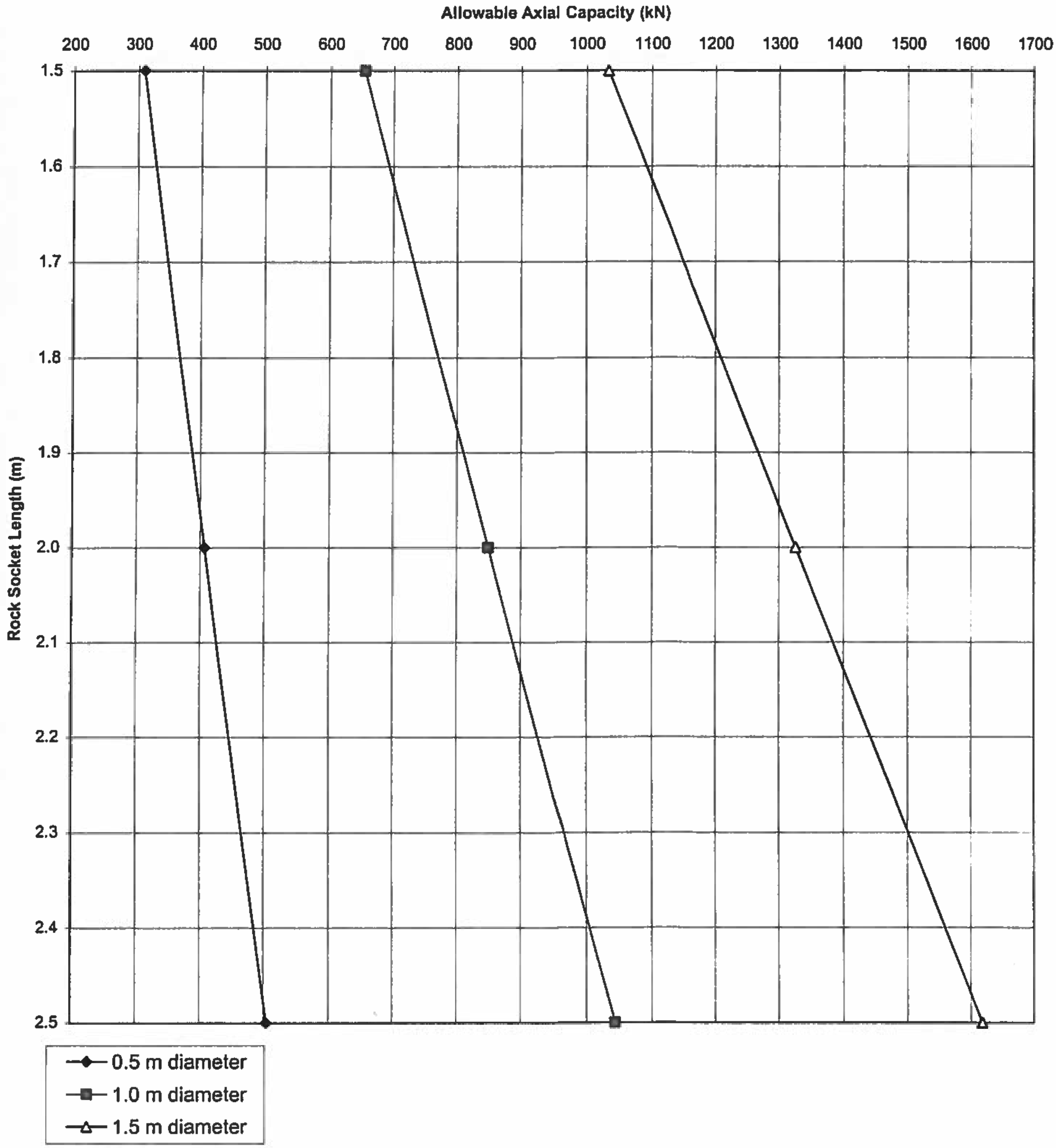



## BORING LOCATION PLAN

Boring Location

Bench Mark

| BIA BRIDGE N651 - TOHATCHI, N.M. | |
|---|---|
| Boring Location Diagram | |
| **Western Technologies Inc.** | |
| Job No.: 2522JW111 | Plate: 2 |




| BIA BRIDGE N651 | |
|---|---|
| Drilled Shaft Capacity | |
| **Western Technologies Inc.** | |
| Job No.: 2522JW111 | Plate: 3 |

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| | |
|---|---|
| **Allowable Soil Bearing Capacity** | The recommended maximum contact stress developed at the interface of the foundation element and the supporting material. |
| **Backfill** | A specified material placed and compacted in a confined area. |
| **Base Course** | A layer of specified material placed on a subgrade or subbase. |
| **Base Course Grade** | Top of base course. |
| **Bench** | A horizontal surface in a sloped deposit. |
| **Caisson** | A concrete foundation element cast in a circular excavation which may have an enlarged base. Sometimes referred to as a cast-in-place pier. |
| **Concrete Slabs-on-Grade** | A concrete surface layer cast directly upon a base, subbase or subgrade. |
| **Crushed Rock Base Course** | A base course composed of crushed rock of a specified gradation. |
| **Differential Settlement** | Unequal settlement between or within foundation elements of a structure. |
| **Engineered Fill** | Specified material placed and compacted to specified density and/or moisture conditions under observations of a representative of a soil engineer. |
| **Existing Fill** | Materials deposited through the action of man prior to exploration of the site. |
| **Existing Grade** | The ground surface at the time of field exploration. |
| **Expansive Potential** | The potential of a soil to expand (increase in volume) due to absorption of moisture. |
| **Fill** | Materials deposited by the actions of man. |
| **Finished Grade** | The final grade created as a part of the project. |
| **Gravel Base Course** | A base course composed of naturally occurring gravel with a specified gradation. |
| **Heave** | Upward movement |
| **Native Grade** | The naturally occurring ground surface. |
| **Native Soil** | Naturally occurring on-site soil. |
| **Rock** | A natural aggregate of mineral grains connected by strong and permanent cohesive forces. Usually requires drilling, wedging, blasting or other methods of extraordinary force for excavation. |
| **Sand & Gravel Base** | A base course of sand and gravel of a specified gradation. |
| **Sand Base Course** | A base course composed primarily of sand of a specified gradation. |
| **Scarify** | To mechanically loosen soil or break down existing soil structure. |
| **Settlement** | Downward movement. |
| **Soil** | Any unconsolidated material composed of discrete solid particles, derived from the physical and/or chemical disintegration of vegetable or mineral matter, which can be separated by gentle mechanical means such as agitation in water. |
| **Strip** | To remove from present location. |
| **Subbase** | A layer of specified material placed to form a layer between the subgrade and base course. |
| **Subbase Grade** | Top of subbase. |
| **Subgrade** | Prepared native soil surface. |

## COARSE-GRAINED SOILS

LESS THAN 50% FINES*

| GROUP SYMBOLS | DESCRIPTION | MAJOR DIVISIONS |
|---|---|---|
| GW | WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LESS THAN 5% FINES | GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE |
| GP | POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LESS THAN 5% FINES | |
| GM | SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, MORE THAN 12% FINES | |
| GC | CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, MORE THAN 12% FINES | |
| SW | WELL-GRADED SANDS OR GRAVELLY SANDS, LESS THAN 5% FINES | SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE |
| SP | POORLY-GRADED SANDS OR GRAVELLY SANDS, LESS THAN 5% FINES | |
| SM | SILTY SANDS, SAND-SILT MIXTURES, MORE THAN 12% FINES | |
| SC | CLAYEY SANDS, SAND-CLAY MIXTURES, MORE THAN 12% FINES | |

NOTE: Coarse-grained soils receive dual symbols if they contain 5% to 12% fines (e.g., SW-SM, GP-GC).

## FINE-GRAINED SOILS

MORE THAN 50% FINES

| GROUP SYMBOLS | DESCRIPTION | MAJOR DIVISIONS |
|---|---|---|
| ML | INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS | SILTS AND CLAYS LIQUID LIMIT LESS THAN 50 |
| CL | INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS | |
| OL | ORGANIC SILTS OR ORGANIC SILT-CLAYS OF LOW PLASTICITY | |
| MH | INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC SILTS | SILTS AND CLAYS LIQUID LIMIT MORE THAN 50 |
| CH | INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS | |
| OH | ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY | |
| PT | PEAT, MUCK AND OTHER HIGHLY ORGANIC SOILS | HIGHLY ORGANIC SOILS |

NOTE: Fine-grained soils may receive dual classification based upon plasticity characteristics.

## SOIL SIZES

| COMPONENT | SIZE RANGE |
|---|---|
| BOULDERS | Above 12 in. |
| COBBLES | 3 in. – 12 in. |
| GRAVEL | No. 4 – 3 in. |
| Coarse | 3/4 in. – 3 in. |
| Fine | No. 4 – 3/4 in. |
| SAND | No. 200 – No. 4 |
| Coarse | No. 10 – No. 4 |
| Medium | No. 40 – No. 10 |
| Fine | No. 200 – No. 40 |
| *Fines (Silt or Clay) | Below No. 200 |

NOTE: Only sizes smaller than three inches are used to classify soils

## CONSISTENCY

| CLAYS & SILTS | BLOWS PER FOOT* |
|---|---|
| VERY SOFT | 0 – 2 |
| SOFT | 2 – 4 |
| FIRM | 4 – 8 |
| STIFF | 8 – 16 |
| VERY STIFF | 16 – 32 |
| HARD | Over 32 |

## RELATIVE DENSITY

| SANDS & GRAVELS | BLOWS PER FOOT* |
|---|---|
| VERY LOOSE | 0 – 4 |
| LOOSE | 4 – 10 |
| MEDIUM DENSE | 10 – 30 |
| DENSE | 30 – 50 |
| VERY DENSE | Over 50 |

*Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 inch ID) split spoon (ASTM D1586).

## PLASTICITY OF FINE GRAINED SOILS

| PLASTICITY INDEX | TERM |
|---|---|
| 0 | NON-PLASTIC |
| 1 – 7 | LOW |
| 8 – 25 | MEDIUM |
| Over 25 | HIGH |

## DEFINITION OF WATER CONTENT

| |
|---|
| DRY |
| SLIGHTLY DAMP |
| DAMP |
| MOIST |
| WET |
| SATURATED |

| | |
|---|---|
| BIA BRIDGE N651 | |
| METHOD OF SOIL CLASSIFICATION | |
| Western Technologies Inc. | |
| Job No.: 2522JW111 | Plate: A-2 |

The number shown in "**BORING NO.**" refers to the approximate location of the same number indicated on the "Boring Location Diagram" as positioned in the field by pacing from property lines and/or existing features.

**"ELEVATION"** refers to ground surface elevation at the boring location established by measurements with a hand level from a bench mark (BM) shown on the "Boring Location Diagram" (Elev. = 1958.5 meters based on contour information provided by ASCG).

**"TYPE BORING"** refers to the exploratory equipment used in the boring wherein **HSA = hollow stem auger, ODEX = downhole percussion, RA = rotary air** and **CNX = NX-size diamond core.**

**"N" in Blows"** refers to the number of blows of a 63.5 kilogram weight, dropped 762 millimeters, required to advance a 51-millimeter-outside-diameter split-barrel sampler a distance of 0.31 meters, Standard Penetration Test (ASTM D1586). Refusal to penetration is defined as more than 100 blows per 0.31 meters.

**"R" in Blows"** refers to the number of blows of a 63.5 kilogram weight, dropped 762 millimeters, required to advance a 61.5 millimeter-inside-diameter ring sampler a distance of 0.31 meters. Refusal to penetration is considered more than 50 blows per 0.31 meters.

**"Sample Type"** refers to the form of sample recovery, in which **N = Split-barrel sample, R = Ring sample, G = Grab Sample and C = Core Run.**

**"Dry Density"** refers to the laboratory-determined dry density in pounds per cubic foot. The symbol "NR" indicates that no sample was recovered. The symbol **"DU"** indicates that determination of dry density was not possible.

**"Water Content, %"** refers to the laboratory-determined moisture content in percent ASTM D2216.

**"RQD"** refers to the Rock Quality Designation index which indicates the recovery of rock cores obtained by adding up the lengths of rock core pieces 10.2 centimeters or longer and dividing this length by the total length of core run.

**"Recovery"** refers to the total length of recovered rock core divided by the length of core run.

**"Unified Classification"** refers to the soil type as defined by "Method of Soil Classification". The soils were classified visually in the field and, where appropriate, classifications were modified by visual examination of samples in the laboratory and/or by appropriate tests.

These notes and boring logs are intended for use in conjunction with the purposes of our services defined in the text. Boring log data should not be construed as part of the construction plans nor as defining construction conditions.

Boring logs depict our interpretations of subsurface conditions at the locations and on the dates noted. Variations in subsurface conditions and soil characteristics may occur between borings. Groundwater levels may fluctuate due to seasonal variations and other factors.

The stratification lines shown on the boring logs represent our interpretation of the approximate boundary between soil types based upon visual field classification. The transition between materials is approximate and may be far more or less gradual than indicated.

| BIA BRIDGE N651 | |
|---|---|
| **BORING LOG NOTES** | |
| **Western Technologies Inc.** | |
| Job No.: 2522JW111 | Plate: A-3 |

DATE DRILLED: 03-05-2002

DRILL RIG TYPE: CME75

BORING TYPE/SIZE: HSA/ODEX/RA

# BORING NO. 1

LOCATION: See Boring Location Diagram

ELEVATION: 1957.37 Meters

FIELD ENGR: WT/ J. SANDOVAL

| WATER CONTENT (%) | DRY DENSITY (LBS/CU.FT) | SAMPLE TYPE | BLOWS/FT. R or N | BLOWS/FT. C | USCS | SOIL DESCRIPTION |
|---|---|---|---|---|---|---|
| | | | | | SM | SILTY SAND; trace gravel, tan, loose, damp, fine to medium grained |
| | | R | 12 | | | |
| | | N | 7 | | | |
| | | R | 10 | | | |
| | | N | 9 | | | |
| | | R | 21 | | | |
| | | N | 11 | | | |
| | | N | 7 | | | groundwater encountered at 7.0 meters |
| | | N | 84 | | GM | GRAVEL AND COBBLES; with sand, tan, medium dense, saturated |
| | NR | N | 50/.05 | | | |
| | | N | 50/.10 | | | SHALE; green to gray, soft to medium hard, damp |
| | | N | 50/.10 | | | |
| | | N | 50/.15 | | | |
| | | N | 50/.13 | | | |
| | | N | 50/.13 | | | |
| | | N | 50/.13 | | | |
| | | N | 50/.13 | | | |
| | | N | 50/.15 | | | |
| | | N | 50/.13 | | | |
| | | N | 50/.08 | | | |
| | | N | 50/.10 | | | |
| | | N | 50/.05 | | | |
| | | | | | | Boring Stopped At 28.3 Meters |

DEPTH (FT.): 5, 10, 15, 20, 25, 30

GROUNDWATER ENCOUNTERED NO: ____ YES: X DEPTH: 7.0 m DATE: 03-05-2002

NOTES HSA to 10.7 meters, ODEX to 12.2 meters

RA to 28.3 meters

BIA BRIDGE N651

Boring Log

Western Technologies Inc.

Job No.: 2522JW111 | Plate: A-4

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

DATE DRILLED: 02-27-2002

DRILL RIG TYPE: CME-75

BORING TYPE/SIZE: HSA/ODEX/RA/CNX

# BORING NO. 2

LOCATION: See Boring Location Diagram

ELEVATION: 1958.01 Meters

FIELD ENGR: WT/ J. Sandoval

| WATER CONTENT (%) | DRY DENSITY (LBS/CU.FT) | SAMPLE TYPE | BLOWS/FT. R or N | BLOWS/FT. C | RQD | RECOVERY (%) | DEPTH (FT.) | USCS | SOIL/ROCK DESCRIPTION |
|---|---|---|---|---|---|---|---|---|---|
| | | G | | | | | | SP | SILTY SAND; trace gravel and cobbles, tan, loose to medium dense, slightly damp to damp, fine grained |
| | | N | 4 | | | | | | |
| | | N | 11 | | | | | | |
| | | N | 18 | | | | 5 | | |
| | NR | N | 49 | | | | | GM | GRAVEL AND SAND; tan, medium dense, damp |
| | | N | 50/.08 | | | | | | large boulder approximately 0.6 meters thick; groundwater encoutered at 8.5 meters |
| | | N | 50/.15 | | | | 10 | | |
| | | N | 59 | | | | | | SHALE; olive to gray, soft, damp |
| | | C | | | 0 | 25 | | | |
| | | N | 50/.15 | | | | | | |
| | | N | 50/.10 | | | | 15 | | |
| | | N | 50/.10 | | | | | | |
| | | N | 50/.15 | | | | | | |
| | | N | 50/.08 | | | | 20 | | Boring Stopped At 19.5 Meters |
| | | | | | | | 25 | | |
| | | | | | | | 30 | | |

GROUNDWATER ENCOUNTERED NO: ____ YES: X DEPTH: 8.5 m DATE: 02-27-2002

NOTES HSA to 9.4 m, ODEX to 10.7 m, RA to 11.6 m, CNX to 13.4 m, RA to 19.5 m

BIA BRIDGE N651

Boring Log

**Western Technologies Inc.**

Job No.: 2522JW111 | Plate: A-5

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

DATE DRILLED: 03-11-2002

DRILL RIG TYPE: CME-75

BORING TYPE/SIZE: HSA/ODEX/RA

# BORING NO. 3

LOCATION: See Boring Location Diagram

ELEVATION: 1955.85 Meters

FIELD ENGR: WT/ J. Sandoval

| WATER CONTENT (%) | DRY DENSITY (LBS/CU.FT) | SAMPLE TYPE | BLOWS/FT. R or N | BLOWS/FT. C |
|---|---|---|---|---|
| | | N | 50/.08 | |
| | | N | 50/.13 | |
| | NR | N | 10 | |
| | | N | 50/.15 | |
| | | N | 32 | |
| | | R | 50/.23 | |
| | | N | 50/.15 | |
| | | N | 50/.13 | |
| | | N | 50/.15 | |
| | | N | 50/.15 | |
| | | N | 50/.13 | |
| | | N | 50/.15 | |
| | | N | 50/.13 | |
| | | N | 50/.15 | |
| | | N | 50/.08 | |
| | | N | 50/.08 | |
| | | N | 50/.13 | |
| | | N | 50/.13 | |
| | | N | 50/.08 | |
| | | N | 50/.08 | |

DEPTH (FT.): 5, 10, 15, 20, 25, 30

| USCS | SOIL DESCRIPTION |
|---|---|
| SM | SILTY SAND; trace gravel, tan, loose, damp |
| GM | GRAVEL AND COBBLES; with sand, tan, medium dense, damp to saturated |
| | groundwater encountered at 5.2 meters |
| | SHALE; auger cuttings classify as clay, green to gray, soft to moderately hard, damp to moist |
| | Boring Stopped At 30 Meters |

GROUNDWATER ENCOUNTERED NO: ____ YES: X DEPTH: 5.2 DATE: 03-11-2002

NOTES HSA to 6.1 meters, ODEX to 9.1 meters

RA to 30 meters

BIA BRIDGE N651

Boring Log

**Western Technologies Inc.**

Job No.: 2522JW111 | Plate: A-6

DATE DRILLED: 03-07-2002

DRILL RIG TYPE: CME-75

BORING TYPE/SIZE: HSA/ODEX/RA

# BORING NO. 4

LOCATION: See Boring Location Diagram

ELEVATION: 1958.38 Meters

FIELD ENGR: WT/ J. Sandoval

| WATER CONTENT (%) | DRY DENSITY (LBS/CU.FT) | SAMPLE TYPE | SAMPLE | BLOWS/FT. R or N | BLOWS/FT. C | DEPTH (FT.) | USCS | GRAPHIC | SOIL DESCRIPTION |
|---|---|---|---|---|---|---|---|---|---|
| | | G | | | | | | | (FILL) SILTY SAND; some gravel, tan, medium dense, slightly damp to damp |
| | | N | | 23 | | | | | |
| | | N | | 86 | | | GP | | COBBLES; some gravel and silty sand, trace boulders, tan, medium dense to dense, damp |
| | | | | | | | SM | | SILTY SAND; tan, medium dense, damp, fine grained |
| | | N | | 50/.08 | | 5 | GP | | GRAVEL; some silty sand, tan, medium dense to dense, slightly damp |
| | | N | | 50/.10 | | | | | |
| | | N | | 25 | | | | | groundwater encountered at 8.5 meters |
| | | N | | 50/.02 | | 10 | | | |
| | | N | | 84 | | | | | |
| | | N | | 50/.10 | | | | | SHALE; green to gray, soft, damp |
| | | N | | 50/.15 | | 15 | | | |
| | | N | | 50/.13 | | | | | |
| | | N | | 50/.13 | | | | | |
| | | N | | 59 | | | | | |
| | | N | | 81 | | 20 | | | |
| | | N | | 50/.13 | | | | | Boring Stopped At 21.7 Meters |
| | | | | | | 25 | | | |
| | | | | | | 30 | | | |

GROUNDWATER ENCOUNTERED NO:____ YES: X DEPTH: 8.5 m DATE: 03-07-2002

NOTES HSA to 9.4 meters, ODEX to 13.7 meters
RA to 21.7 meters

BIA BRIDGE N651

Boring Log

**Western Technologies Inc.**

Job No.: 2522JW111 | Plate: A-7

B

## PHYSICAL PROPERTIES

| BORING NO. | DEPTH (METERS) | SOIL CLASSIFICATION | PARTICLE SIZE DISTRIBUTION % PASSING BY WEIGHT | | | | | ATTERBERG LIMITS | | MINIMUM-MAXIMUM DENSITY | | 'R' VALUE | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 76.2 mm | NO. 4 | NO. 10 | NO. 40 | NO. 200 | LL | PI | MINIMUM DENSITY (KG/M³) | MAXIMUM DENSITY (KG/M³) | | |
| 1 | 4.7-5.0 | SM | | 100 | 98 | 93 | 40 | | NP | | | | 2 |
| 2 | 0-1.5 | SP-SM | | | 100 | 98 | 12 | | NP | 1392 | 1744 | | 2,6 |
| 3 | 9.3-9.5 | Clayshale | | | 100 | 97 | 79 | 50 | 30 | | | | 2 |
| 4 | 0-1.5 | SM | 100 | 98 | 97 | 95 | 33 | 23 | 3 | | | | 2 |

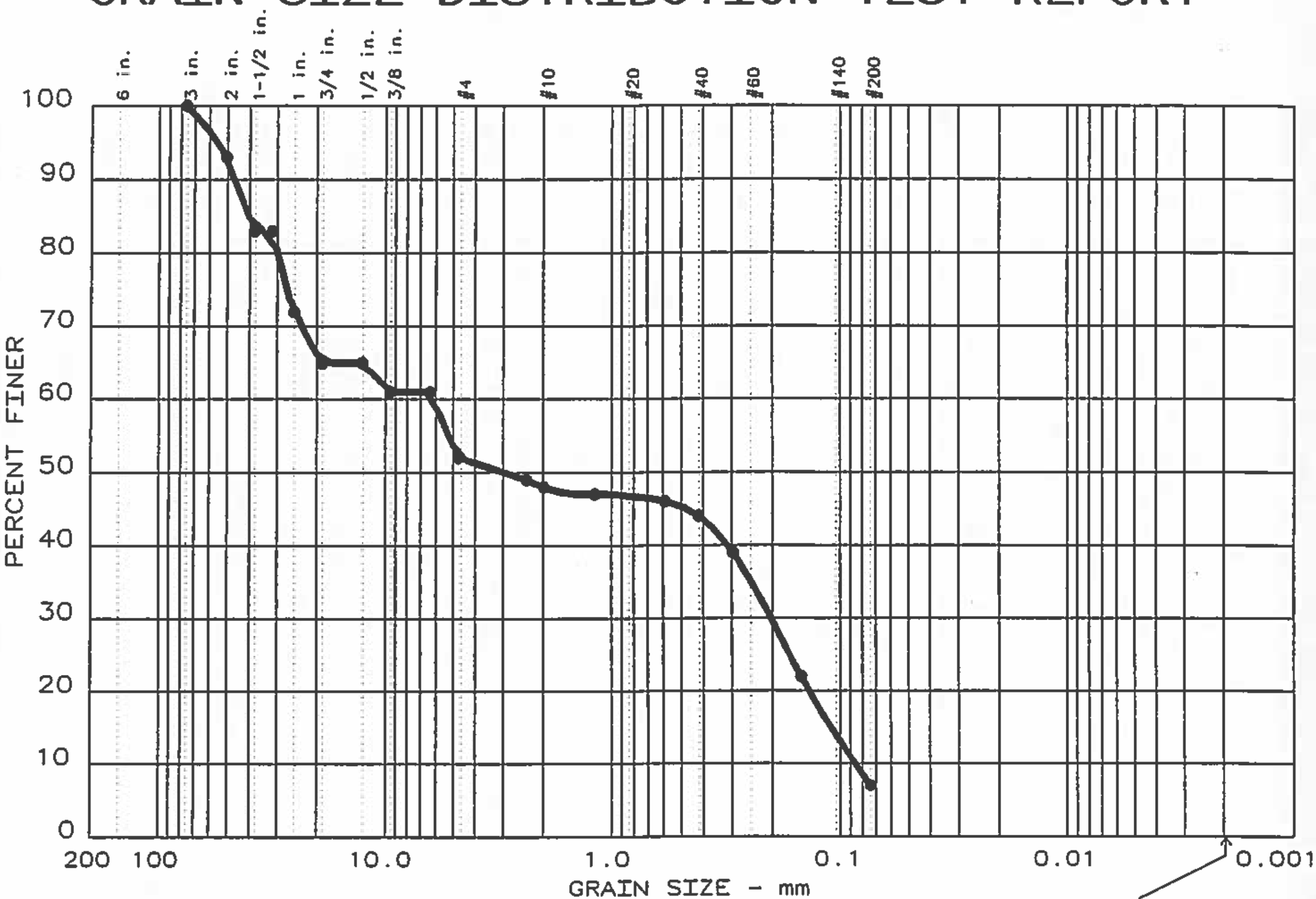
**REMARKS:**
**CLASSIFICATION / PARTICLE SIZE**
1. Visual
2. Laboratory Tested
3. Minus No. 200 Only

**MOISTURE-DENSITY RELATIONSHIP**
4. Tested ASTM D698 / AASHTO T99
5. Tested ASTM D1557 / AASHTO T180
6. Tested ASTM D4253 / D4254

**NOTE:** NP Nonplastic

| BIA BRIDGE N651 | |
|---|---|
| PHYSICAL PROPERTIES | |
| **Western Technologies Inc.** | |
| Job No.: 2522JW111 | Plate: B-1 |

# GRAIN SIZE DISTRIBUTION TEST REPORT




| | Test | % +3" | % GRAVEL | % SAND | % SILT | % CLAY |
|---|---|---|---|---|---|---|
| ● | 4 | 0.0 | 48.0 | 45.0 | 7.0 | |
| | | | | | | |
| | | | | | | |

| | LL | PI | $D_{85}$ | $D_{60}$ | $D_{50}$ | $D_{30}$ | $D_{15}$ | $D_{10}$ | $C_c$ | $C_u$ |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | | NP | 40.27 | 6.14 | 2.98 | 0.203 | 0.1089 | 0.0855 | 0.08 | 71.9 |
| | | | | | | | | | | |
| | | | | | | | | | | |

| MATERIAL DESCRIPTION | USCS | AASHTO |
|---|---|---|
| ● poorly graded gravel with silt and sand | GP-GM | A-1-b(0) |

Project No.: 2522JW111
Project: BIA BRIDGE N651
● Location: channel bed at crossing

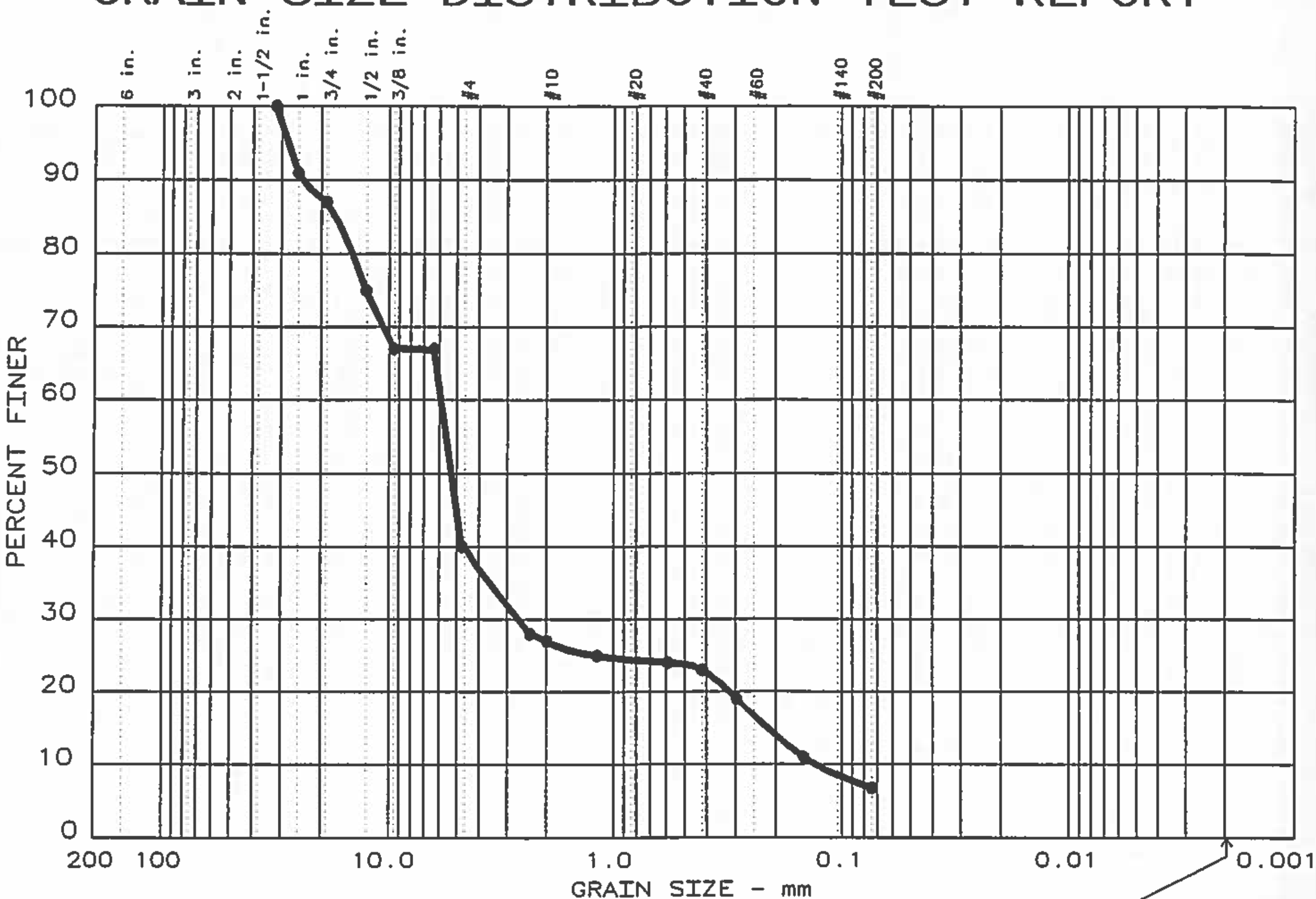
Date: 6/24/02

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
**WESTERN TECHNOLOGIES INC.**

Figure No. B-2

# GRAIN SIZE DISTRIBUTION TEST REPORT




| | Test | % +3" | % GRAVEL | % SAND | % SILT | % CLAY |
|---|---|---|---|---|---|---|
| ● | 5 | 0.0 | 60.0 | 33.2 | 6.8 | |
| | | | | | | |
| | | | | | | |

| | LL | PI | $D_{85}$ | $D_{60}$ | $D_{50}$ | $D_{30}$ | $D_{15}$ | $D_{10}$ | $C_c$ | $C_u$ |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | | NP | 17.18 | 5.89 | 5.29 | 2.652 | 0.2155 | 0.1314 | 9.09 | 44.8 |
| | | | | | | | | | | |
| | | | | | | | | | | |

| MATERIAL DESCRIPTION | USCS | AASHTO |
|---|---|---|
| ● poorly graded gravel with silt and sand | GP-GM | A-1-a(0) |

Project No.: 2522JW111
Project: BIA BRIDGE N651
● Location: channel bed - 60 meters upstream

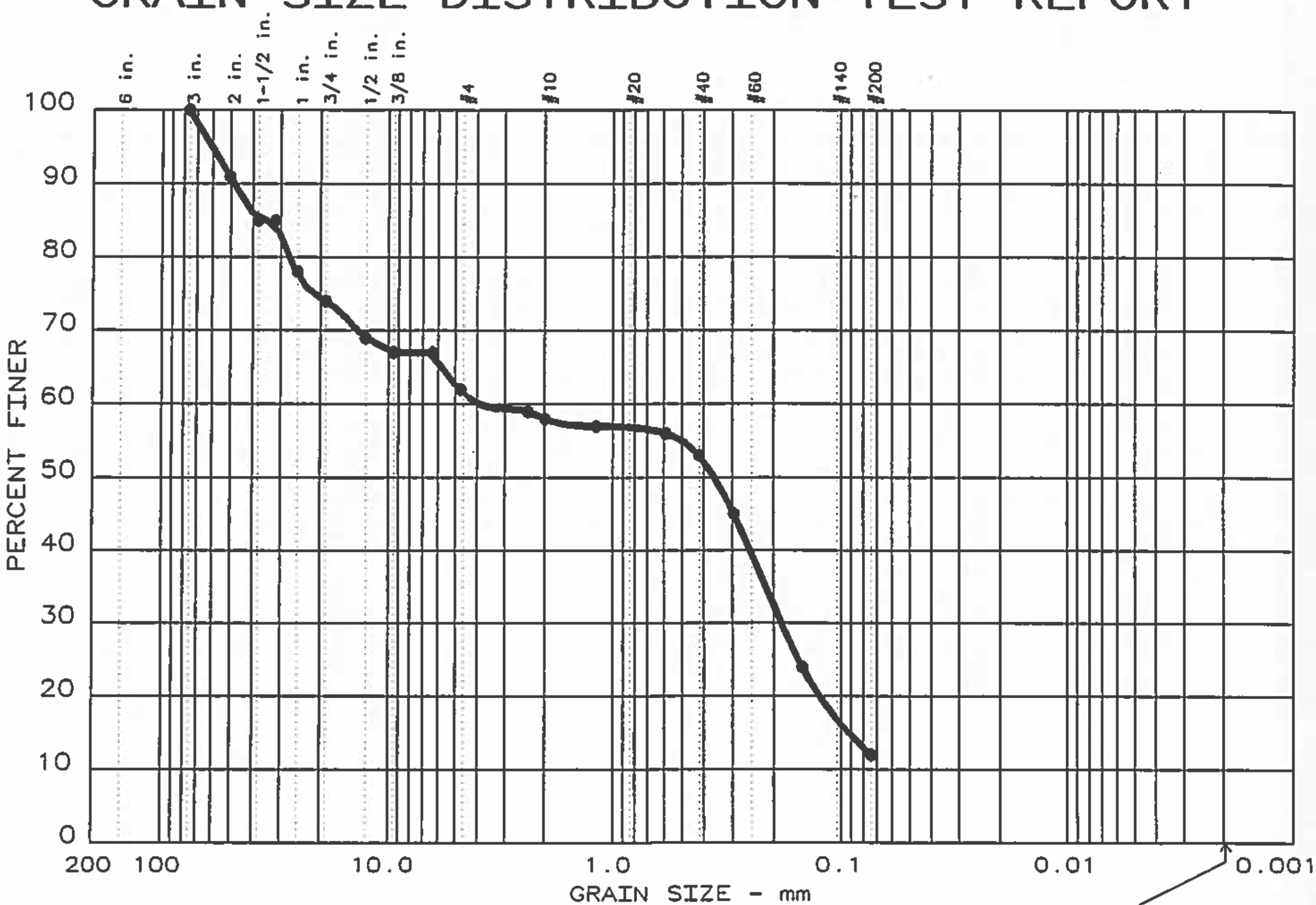
Date: 6/24/02

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
**WESTERN TECHNOLOGIES INC.**

Figure No. B-3

# GRAIN SIZE DISTRIBUTION TEST REPORT




| | Test | % +3" | % GRAVEL | % SAND | % SILT | % CLAY |
|---|---|---|---|---|---|---|
| ● | 6 | 0.0 | 38.0 | 50.0 | 12.0 | |
| | | | | | | |
| | | | | | | |

| | LL | PI | $D_{85}$ | $D_{60}$ | $D_{50}$ | $D_{30}$ | $D_{15}$ | $D_{10}$ | $C_c$ | $C_u$ |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | | NP | 31.62 | 3.89 | 0.36 | 0.184 | 0.0923 | | | |
| | | | | | | | | | | |
| | | | | | | | | | | |

| MATERIAL DESCRIPTION | USCS | AASHTO |
|---|---|---|
| ● poorly graded sand with silt and gravel | SP-SM | A-2-4(0) |

Project No.: 2522JW111
Project: BIA BRIDGE N651
● Location: channel bed - 60 meters downstream

Date: 6/24/02

GRAIN SIZE DISTRIBUTION TEST REPORT
**WESTERN TECHNOLOGIES INC.**

Remarks:

Figure No. B-4

## SOIL PROPERTIES

| BORING NO. | DEPTH (METERS) | SOIL CLASSIFICATION | SOIL PROPERTY: INITIAL DRY DENSITY (KG/M³) | SOIL PROPERTY: INITIAL WATER CONTENT (%) | SHEAR STRENGTH: C (kPa) | SHEAR STRENGTH: Ø (DEGREES) | PERMEABILITY: K (CM/SECOND) | SPECIFIC GRAVITY | WATER SOLUBLE MATTER (PPM): SALTS | WATER SOLUBLE MATTER (PPM): SULFATES | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1.2-1.5 | SM | -- | -- | 10.1 | 35 | | | | | DS, 5 |
| 1 | 2.7-3.0 | SM | -- | -- | 9.6 | 31 | | | | | DS, 5 |
| 1 | 4.3-4.6 | SM | -- | -- | 20.3 | 32 | | | | | DS, 5 |
| 1 | 4.7-5.0 | SM | | | | | | | 259 | 161 | |
| 3 | 8.8-9.1 | clayshale | -- | -- | 43.1 | 11 | | | | | DS, 5 |
| 3 | 9.3-9.5 | clayshale | | | | | | | 127 | 10 | |
| 4 | 0-1.5 | SM | | | | | | | 642 | 295 | |

**NOTE:** Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.

**LEGEND:**

**SHEAR STRENGTH TEST METHOD**

DS Direct Shear
DS Direct Shear (Saturated)
UC Unconfined Compression
UU Unconsolidated Undrained
CU Consolidated Undrained with Pore Pressure
CU Consolidated Undrained
CD Consolidated Drained

**REMARKS:**

1. Compacted Density (approximately 95% of ASTM D698 at moisture value slightly below optimum).
2. Visual Classification.
3. Constant Head.
4. Falling Head.
5. In-Situ Moisture/Density Not Determined.

BIA BRIDGE N651

SOIL PROPERTIES

**Western Technologies Inc.**

Job No.: 2522JW111 | Plate: B-5

# PHYSICAL PROPERTIES

| BORING NO. | DEPTH (METERS) | SOIL CLASSIFICATION | PARTICLE SIZE DISTRIBUTION % PASSING BY WEIGHT | | | | | ATTERBERG LIMITS | | CORROSIVITY | | 'R' VALUE | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 76.2 mm | NO. 4 | NO. 10 | NO. 40 | NO. 200 | LL | PI | pH | RESISTIVITY (OHM-CM) | | |
| 4 | 0-1.5 | SM | | | | | | | | 7.53 | 2168 | | |

**REMARKS:**
**CLASSIFICATION / PARTICLE SIZE**
1. Visual
2. Laboratory Tested
3. Minus No. 200 Only

**MOISTURE-DENSITY RELATIONSHIP**
4. Tested ASTM D698 / AASHTO T99
5. Tested ASTM D1557 / AASHTO T180

**NOTE: NP** Nonplastic

| BIA BRIDGE N651 | |
|---|---|
| **PHYSICAL PROPERTIES** | |
| **Western Technologies Inc.** | |
| Job No.: 2522JW111 | Plate: B-6 |

C

$$q_{ult} = cN_c + 0.5\gamma BN_\gamma + qN_q \quad (4.4.7.1\text{-}1)$$

where: $q_{ult}$ = ultimate bearing capacity for uniform pressure
$c$ = soil cohesion = 9.58 kPa (from laboratory testing)
$\gamma$ = total soil unit weight = 1762.0 kg/m$^3$ (assumed)
$B$ = width of footing
$q$ = effective overburden pressure at bottom of footing
for $\phi$ = 31°:
$N_c$ = 32.67
$N_\gamma$ = 25.99
$N_q$ = 20.63
bearing capacity factors (Table 4.4.7.1A)

$$q_{all} = q_{ult}/FS \quad (4.4.7.1\text{-}2)$$

where: $q_{all}$ = allowable uniform bearing pressure
FS = 3.0 (Article 4.4.7.1.2)

| Footing Depth (m) | Footing Width (m) | $q_{ult}$ (kg/m$^2$) | $q_{all}$ (kg/m$^2$) |
|---|---|---|---|
| 1.0 | 1.0 | 91,149.18 | 30,383.06 |
| | 1.5 | 102,597.77 | 34,199.26 |
| | 2.0 | 114,046.37 | 38,015.46 |
| | 2.5 | 125,494.96 | 41,831.65 |
| 1.5 | 1.0 | 109,324.21 | 36,441.40 |
| | 1.5 | 120,772.80 | 40,257.60 |
| | 2.0 | 132,221.40 | 44,073.80 |
| | 2.5 | 143,669.99 | 47,890.00 |
| 2.0 | 1.0 | 127,499.24 | 42,499.75 |
| | 1.5 | 138,947.83 | 46,315.94 |
| | 2.0 | 150,396.43 | 50,132.14 |
| | 2.5 | 161,845.02 | 53,948.34 |

ref: AASHTO *Standard Specifications for Highway Bridges*, 16th Edition

| BIA BRIDGE N651 | |
|---|---|
| **Bearing Capacity Calculations** | |
| **Western Technologies Inc.** | |
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$S_t = S_e + S_c + S_s$ (4.4.7.2-1)

where: $S_t$ = total settlement
$S_e$ = elastic settlement
$S_c$ = consolidation settlement = 0 (granular soils)
$S_s$ = secondary settlement = 0 (granular soils)

$S_e = [q_o(1-\nu^2)(A^{1/2})]/E_s\beta_z$ (4.4.7.2.2-1)

where: $q_o$ = $q_{all}$ = applied vertical stress at bottom of footing
$\nu$ = Poisson's ratio = 0.3 (Table 4.4.7.2.2A)
A = contact area of footing
$E_s$ = Young's Modulus = 28,728.16 kPa (Table 4.4.7.2.2A)
$\beta_z$ = elastic shape/rigidity factor (Table 4.4.7.2.2B)

$D_f$ = footing depth
B = footing width
Assume footing length: L = 10.06 meters

| $D_f$ (m) | B (m) | $q_o$ (kg/m$^2$) | L/B | $\beta_z$ | A (m$^2$) | $S_e$ (cm) |
|---|---|---|---|---|---|---|
| 1.0 | 1.0 | 30383.1 | 10.1 | 1.41 | 10.1 | 2.12 |
| | 1.5 | 34199.3 | 6.7 | 1.35 | 15.1 | 3.06 |
| | 2.0 | 38015.5 | 5.0 | 1.28 | 20.1 | 4.14 |
| | 2.5 | 41831.7 | 4.0 | 1.24 | 25.1 | 5.25 |
| 1.5 | 1.0 | 36441.4 | 10.1 | 1.41 | 10.1 | 2.55 |
| | 1.5 | 40257.6 | 6.7 | 1.35 | 15.1 | 3.60 |
| | 2.0 | 44073.8 | 5.0 | 1.28 | 20.1 | 4.80 |
| | 2.5 | 47890.0 | 4.0 | 1.24 | 25.1 | 6.02 |
| 2.0 | 1.0 | 42499.7 | 10.1 | 1.41 | 10.1 | 2.97 |
| | 1.5 | 46315.9 | 6.7 | 1.35 | 15.1 | 4.14 |
| | 2.0 | 50132.1 | 5.0 | 1.28 | 20.1 | 5.46 |
| | 2.5 | 53948.3 | 4.0 | 1.24 | 25.1 | 6.78 |

ref: AASHTO *Standard Specifications for Highway Bridges*, 16th Edition

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|---|---|
| **Settlement Calculations** | |
| **Western Technologies Inc.** | |
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D

$Q_{all} = A_T F_a$ (Article 4.5.7)

where: $Q_{all}$ = allowable axial load capacity
$A_T$ = cross-section area of pile
$F_a$ = $0.25F_Y$ = allowable steel stress
$F_Y$ = minimum yield stress of steel = 248.2 MPa

| Pile Section | $A_T$ ($m^3$) | $Q_{all}$ (kN) |
|---|---|---|
| HP 10x42 | 0.008 | 496.4 |
| HP 12x53 | 0.010 | 620.5 |
| HP 12x74 | 0.014 | 872.7 |
| HP 14x102 | 0.019 | 1201.0 |

ref: AASHTO *Standard Specifications for Highway Bridges*, 16th Edition
NAVFAC Design Manual 7.2

| BIA BRIDGE N651 | |
|---|---|
| **Driven Pile Capacity Calculations** | |
| **Western Technologies Inc.** | |
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$Q_{SR} = \pi B_r D_r q_{SR}$ (4.6.5.3.1-1)

where: $Q_{SR}$ = ultimate side resistance of rock socket
$B_r$ = diameter of rock socket
$D_r$ = length of rock socket
$q_{SR}$ = ultimate unit shear resistance along shaft/rock interface

$C_M = \alpha_E C_O$ (4.6.5.3.1-2)

where: $C_M$ = uniaxial compressive strength of rock mass
$\alpha_E$ = $0.0231(RQD) - 1.32 \geq 0.15$ (4.4.8.2.2-4)
$C_O$ = uniaxial compressive strength of rock = 13.8 MPa (Table 4.4.8.1.2B for shale)

$\alpha_E$ = 0.15
$C_M$ = 0.15(13.8) = 2.07 MPa
$q_{SR}$ = 310.3 kPa with $C_M$ = 2.07 MPa (Fig. 4.6.5.3.1A)

$Q_{TR} = N_{MS} C_O A_T$ (4.6.5.3.2-1)

where: $Q_{TR}$ = ultimate tip resistance of rock socket
$N_{MS}$ = 0.016 for Rock Category B with poor rock mass quality (Table 4.4.8.1.2A)
$C_O$ = 16.18 MPa
$A_T$ = area of shaft tip

$Q_{all} = (Q_{SR} + Q_{TR})/FS$

where: $Q_{all}$ = allowable axial load capacity
FS = 2.5 (Article 4.6.5.4)

ref: AASHTO *Standard Specifications for Highway Bridges*, 16th Edition

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|---|---|
| **Drilled Shaft Capacity Calculations** | |
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| $B_r$ (m) | $A_T$ ($m^2$) | $D_r$ (m) | $Q_{SR}$ (kN) | $Q_{TR}$ (kN) | $Q_{all}$ (kN) |
|---|---|---|---|---|---|
| 0.5 | 0.196 | 1.5 | 731.1 | 43.4 | 309.8 |
| | | 2.0 | 974.8 | 43.4 | 407.3 |
| | | 2.5 | 1218.5 | 43.4 | 504.8 |
| 1.0 | 0.785 | 1.5 | 1462.3 | 173.4 | 654.3 |
| | | 2.0 | 1949.7 | 173.4 | 849.2 |
| | | 2.5 | 2437.1 | 173.4 | 1044.2 |
| 1.5 | 1.767 | 1.5 | 2193.4 | 390.2 | 1033.4 |
| | | 2.0 | 2924.5 | 390.2 | 1325.9 |
| | | 2.5 | 3655.6 | 390.2 | 1618.3 |

| BIA BRIDGE N651 | |
|---|---|
| Drilled Shaft Capacity Calculations | |
| **Western Technologies Inc.** | |
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