TECHNICAL MEMORANDUM

Date: March 23, 2021

To: City of Spokane Planning Department

From: Jerry Storhaug, P.E.
Storhaug Engineering
510 East Third Avenue
Spokane, WA 99202

Project: Storhaug Project 19-087 Corbin Cottages Preliminary Subdivision

Subject: **Corbin Cottages 13-Lot Preliminary Subdivision: Sewer, Water, and Stormwater Concept Design**

EXECUTIVE SUMMARY

This technical memorandum describes the design concept to provide the proposed 13-lot subdivision with sanitary sewer collection, water distribution for domestic use and fire protection, and stormwater general concept infrastructure. This property is located on Spokane Co. Assessor's Parcel No. 35064.3611. Sewer and water mains will be constructed within the public road system and stubbed to the individual lot areas throughout the project. The connection point for both sewer and water is at the entry to the project on Cora Avenue. Stormwater infrastructure will consist of Spokane typical grassy swales with drywell infiltration of collected peak storm runoff.

The designs discussed here are conceptual. A geotechnical report will be provided as part of the construction drawing submittal.

PROJECT INFORMATION

Existing zoning/units per acre	RSF-~~C~~/4-10 proposed zoning RSF-C/4-10 units per acre
Lot size range	4,550 s.f. To 5,280 s.f.
Tract size	1,228 s.f.
No. Of parcels	14 (includes tract 'a')
No. Of dwelling units	13 dwelling units
Total plat area 1.89 acres - sites less than or equal t0 2 acres do not require transition lot sizes, per SMC 7c.110.200(c)(1).	
Public R.O.W. Area	0.47 acres
Net lot area	1.40 acres
Net density 9.29 units per net acre	
Existing structures/uses	n/a
Sanitary sewer	City Of Spokane

Water purveyor	City Of Spokane
Proposed uses	Residential
Topographic information	0% to 5% slopes in areas of proposed disturbance
Max. Roof height limit	35'
R.O.W.. Width 50' - residential standard proposed with sidewalks in easements, per smc table 17h.010-1.	
Building setbacks:	
front	15' building/20' garage
rear	15' in RSF-C zone
side/flanking	5'

EXISTING CONDITIONS

Previous investigations by Gifford Consultants indicate that this area is the site was the source of for gravel and sand borrow. Reclamation of the previous mining area has been reported to have included uncontrolled fill which included demolition debris, soil, and trash. In the early 70's the site was developed as a manufactured home park. This use existed for approximately 20 years to the mid 90's. Presently the site area is vacant, with a covering of weeds and grasses, over a surface base of dirt and gravel. Adjoining and to the east of the site is the Faith Bible Church. The site has a USDA NRCS classification of Spens very gravelly loamy coarse sand and Urban land Opportunity disturbed complex. Previous soil reporting indicate that the existing soils / fill general consist of gravelly sand to sandy gravel, with cobbles, boulders and variable quantities of trash and debris. The depth to groundwater is estimated to be greater than 200 feet. Mitigation recommendations were made for the adjacent Faith Bible Church prior to construction. These measures left the existing soils in place with a preloading of soils. Following these mitigations, typical construction methods were utilized including a permeability recommendation of 10 to 100 feet per day. Typical drywell / swale methods for pretreatment and infiltration of storm water were recommended. A copy of the Gifford Consultants Geotechnical Report is included with this technical memorandum submittal. As part of the preparation of the infrastructure construction documents, the previous geotechnical report will be amended to address construction mitigations, and recommendations for this specific site.

SANITARY SEWER

The point of connection for the sanitary sewer will be a new manhole built over the existing 8" concrete sewer line at the southern boundary of the site. The existing sewer main in Cora Avenue is approximately 8 feet deep. 8-inch PVC sewer mains are proposed to be construction throughout the site to carry the wastewater to the point of connection. The system will be designed per City of Spokane Design Standards, and all platted lots are intended to be served by gravity. A dry sewer manhole will be constructed at the terminus northern point of the public sewer line, constructed within the public street. The sewer man will be between 9 to 11 feet in depth. All construction will be in accordance with the City of Spokane public works standards.

WATER DISTRIBUTION

The point of connection for the project is the 6-inch cast iron water main at the southern boundary of the site. The distribution to the individual platted lots will take place behind a individual meter located on each lot. The water main located on the proposed property will be ductile iron. The water system will be public and will be designed and constructed per City of Spokane standards. Per the City of Spokane, the static pressure at the point of connection is approximately 72 psi.

A fire hydrant will be installed at the northern end of the new street. This length is less than 400 feet from existing line in Cora Line. Per the predevelopment meeting notes, we are expecting a fire flow requirement of 1000 gpm. At this flow rate, we are anticipating that fire flow line friction and head losses to be less than 20 feet. The existing pressure and water network will support this flow rate.

STORM WATER TREATMENT AND CONTROL

Storm water and surface drainage will be disposed onsite in accordance with SMC 17D.060.140 and following the requirements of the Spokane Regional Stormwater Manual. As part of the preparation of the construction plans a geotechnical report will be provided. For the purpose of this concept report we are assuming that encountered conditions will be similar to as found on the adjoining church site. We are anticipating well-draining granular soils, which are characteristic of this area. NCRS mapping indicates that the site is underlain with moderately deep, well drained soils formed in colluvium and residuum weathered from granite, gneiss and schist mixed with loess and volcanic ash in the upper part. These soils are well-drained with high saturated hydraulic conductivity. It should be repeated that this is site is a reclaimed gravel pit. Test pit explorations in or near this area indicated that the fill was primarily granular sands and gravels. Whenever possible it is intended to utilize the DOE low impact development guidance manual. Drainage swales between the curb line and the sidewalk are intended. The roadside swales will provide primary treatment of collected stormwater. Drywells and subsurface galleries will be used for subsurface disposal of collected stormwater during peak stormwater events. An erosion and sediment control plan will be prepared as part of the infrastructure plans submitted at the time of final plat approval.

Sincerely,

Jerry Storhaug, PE

Encl. Gifford Consultants Geotechnical Report

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E-1487-02

October 6, 1994

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Attn: Mr. Walt Miller, President

REPORT OF SUBSURFACE EXPLORATIONS AND GEOTECHNICAL ENGINEERING STUDIES FOR THE PROPOSED NEW FAITH BIBLE CHURCH, SPOKANE, WASHINGTON

This letter report presents the results of subsurface explorations and geotechnical engineering studies that were conducted for the proposed new Faith Bible Church that will be constructed in Spokane, Washington. The purpose of this work was to provide site specific, subsurface data in order to develop recommendations for design and construction of foundations, pavements, and related earthwork.

The work was accomplished in general accordance with our proposal letter, dated August 2, 1994, which was authorized by the return of a signed copy on August 18, 1994.

SITE AND PROJECT DESCRIPTIONS

We understand that Faith Bible Church plans to construct a new church building on a 20 acre site, located between Glass and Cora Avenues, approximately 1,000 to 3,000 feet west of Division Street, as shown on the vicinity map, Fig. 1. Most of the southern and central parts of the site are relatively level, sloping down gently to the south. The northern and eastern edges of the site consist of a steep bank, sloping down to the south and southwest at about 70 percent.

Until recently, the site was occupied by a mobile home park, consisting of several paved streets, a wood-frame community center building, and spaces for approximately 200 mobile homes. The site has various underground utilities, including water, natural gas, electric power, sanitary sewer, and cable television. We understand that most of the underground utility lines have been abandoned. Only one mobile home remains on the site.

Historical data indicates that the site and adjacent areas were formerly used for many years as a source for gravel and sand borrow. A series of large borrow pits was excavated to about 60 to 70 feet deep in this area. The City of Spokane used the pits from 1953 until 1954 for the disposal of solid waste. In the 1950's and 1960's, the pits were partially filled by others with demolition debris, soil, and trash. In the early 1970s, filling was completed to the present grade and, in 1974, the trailer park was constructed.

The proposed new building will be located in the central part of the site, as shown on Fig. 2. It will consist of an irregularly shaped, one-story, slab-on-grade structure, with a footprint area of approximately 30,000 square feet. The project also includes constructing several acres of paved parking and driveways. Because the building design is still in the preliminary stages, the structural loads are not yet known. However, you estimated that wall loads in the sanctuary could range up to 5 klf and that wall loads in the remainder of the building could range from about 2 to 2.5 klf. Individual column loads could range up to about 50 kips each.

We understand that the finish floor elevation of the new building will be approximately 1949 feet (City of Spokane Data). This is approximately 1 to 3 ft. higher than the existing ground surface.

Criteria for pavement design recommendations are based on an assumed traffic index of 4.0 for parking areas, 5.0 for driveways, and a 20-year pavement life cycle.

PREVIOUS STUDY

In 1991, Gifford Consultants, Inc., conducted subsurface explorations and made a preliminary geotechnical engineering assessment of the site. Results of this work were presented in a letter report, dated September 27, 1991, to Great Western Savings Bank. Subsurface explorations included making four hollow-stem auger borings and eleven backhoe test pits. Locations of the previous borings and test pits that are close to the proposed new building area are shown on Fig. 2 as the 100 and 200 series explorations. The 1991 studies included reviewing previous environmental reports and analyzing U.S.G.S. topographic maps.

The 1991 explorations encountered up to about 60 feet of very loose to dense fill soil, overtop of native gravelly sand. The fill consisted of gravelly SAND to sandy GRAVEL and contained cobbles, boulders, and variable quantities of trash and debris. Based on this information, we concluded that new buildings constructed overtop of the existing fill and supported on shallow footings could experience substantial differential settlement. The 1991 explorations, which were spaced about 150 to 300 ft. apart, suggested that there were two possible areas on the site, one located near the center and one on the east end, where it appeared that the fill was probably less than about 10 feet thick.

CURRENT SUBSURFACE EXPLORATIONS

To evaluate subsurface conditions for the proposed new building location, nine exploratory test pits and six borings, identified as the 300 series and 400 series explorations, respectively, were made at the approximate locations shown on Fig. 2.

The exploratory test pits were excavated on July 15, 1994, with a rubber-tired, tractor-mounted, hydraulic backhoe provided by Vietzke Excavating, a local excavating contractor working under subcontract to our firm. Individual pits ranged from 8 to 15 feet deep. Six of the test pits were terminated at depths of about 8 to 10 feet because of caving. Two test pits were terminated at depths of 14 to 15 feet because of the limitations of the backhoe.

Soil conditions in the exploratory test pits were visually classified as they were exposed by Mr. David Phelps, our geotechnical engineer. Grab samples for laboratory testing were collected from selected soil units exposed in the pit sidewalls. Summary logs of the test pits are presented in Table 1.

Upon completing the test pit excavations and logging, the pits were backfilled with the soil that had been excavated. The backfill was tamped in layers with the backhoe bucket to achieve a moderate degree of compaction.

Because we initially estimated that the existing FILL in the building area was only about 10 ft. thick, we believed that test pit explorations would be appropriate. However, because only one of the test pit excavations completely penetrated the existing FILL, the exploration program was modified to include six borings, drilled at the approximate locations shown on Fig. 2.

Borings ranged from 25.5 to 65.5 feet deep. The drilling totaled 240.5 feet and was accomplished during the period of August 24 through August 29, 1994, using a CME-75, truck-mounted drill rig. The borings were advanced using 7-1/4 in. O.D., 3-3/4 in. I.D., hollow-stem auger.

Representative soil samples were obtained at 2.5 to 5.0 foot depth intervals, using a 2 in. O.D., split-spoon drive sampler. Standard Penetration Tests were performed in conjunction with the drive sampling. These tests (ASTM D 1586) involve driving the sampler a total of 18 inches with a 140 pound drop hammer, freely falling a distance of 30 inches. In performing these tests, the sampler is driven through three successive 6 inch increments of penetration. The sum of the number of blows for the last two increments, that is, the last foot of penetration, is defined as the Standard Penetration Resistance, or N-value. This value is a widely accepted, empirical parameter that can be approximately correlated to certain engineering characteristics of the soils sampled.

Sample recovery in the existing FILL was locally poor. Consequently, subsurface data in some intervals is sketchy. We believe that the poor recovery was caused by cobble and gravel sized fragments being pushed by the sampler head through loose soils and being too large to enter the sampler.

Drilling and sampling operations were observed and recorded by Mr. Brian Binsfield, our field geotechnical engineer. Mr. Binsfield collected and field classified samples and developed detailed field boring logs. Split-spoon samples were sealed in jars to preserve natural moisture and were returned to our laboratory.

Logs of the borings are presented in Figs. 3 through 8. Summary descriptions of the soil units encountered in the borings, based on our interpretation of the data from the field and laboratory inspections, are shown on the logs, along with plots of Standard Penetration Test N-values and results of moisture content tests.

Upon completing the drilling, the borings were backfilled with cuttings and the upper 3 feet was tamped to minimize settlement. Test pit and boring locations were determined by taping from the building corner stakes, which were laid out by the project surveyor. Approximate ground surface elevations were estimated from elevation data shown on a topographic site plan provided by the previous project surveyor.

LABORATORY TESTING

Soil samples were classified visually as they were recovered. Upon receipt in the laboratory, samples were reexamined to verify and refine field classifications, in general accordance with the procedures described in ASTM D 2488.

Natural moisture contents were determined on all recovered samples to aid in classifying the soil and evaluating engineering properties. Moisture contents are expressed as a percentage, based on the dry weight of the samples. Graphic plots of moisture content vs. depth are shown on the respective boring logs, Figs. 3 through 8. Grain size analyses (ASTM D 422) were conducted on four samples to correlate the field and laboratory visual classifications and for use in describing the soil units. Test results are presented on Figs. 9 and 10.

It should be noted that because of difficulties in obtaining representative samples of coarse grained soils, the gradations shown on the grain size classification curves may not accurately reflect the actual in-place soil gradation. The sampler size limits the maximum particle size to 1 inch. Therefore, larger particle sizes are not represented.

After the laboratory work was completed, the samples were resealed to preserve moisture and stored.

SUBSURFACE INTERPRETATION

Based on the 1991 data and the current boring and test pit explorations, the subsurface conditions at this site consist primarily of existing FILL soil, overlying medium to coarse grained alluvial sediments. Generally, two distinct soil units were encountered in the borings and exploratory test pits conducted for this project:

- Existing FILL
- Native, gravelly SAND

These units can be described in general terms, as follows.

Existing FILL was encountered at the ground surface in all borings and test pits. Recovered samples show that the FILL is variable in consistency and composition, but typically consists of very loose to dense, light brown to dark brown, clean to organic, slightly silty to silty gravelly SAND to sandy GRAVEL; with variable percentages of cobbles, boulders and debris (brick, coal, glass, steel cans, ash, wood, wire, pipe, concrete, etc.).

Standard Penetration Test N-values ranged from 2 to 72 blows/ft. and averaged about 15 blows/ft. The natural moisture content ranged from 1 to 11 percent and averaged about 4 percent.

Native, gravelly SAND was encountered below the existing FILL in most of the borings and one of the test pits. This soil unit was described as consisting of medium dense to very dense, brown to gray, clean to slightly silty, gravelly SAND to sandy GRAVEL, containing cobbles and boulders. Standard Penetration Test N-values ranged from 12 to 76 blows/ft. and averaged about 35 blows/ft. Natural moisture contents ranged from about 1 to 5 percent and averaged about 2 percent.

The stratigraphic relationship between the existing FILL and native gravelly SAND is shown on the approximate subsurface profile, included as Fig. 11. This profile shows that the existing FILL ranges up to about 60 ft. thick.

Ground water was not encountered in any of the borings or test pits for this project. However, we understand that ground water levels in monitoring wells installed by others in 1989 ranged from about 71 to 77 ft. below the existing ground surface.

CONCLUSIONS AND RECOMMENDATIONS

General Comments

Subsurface explorations show that the building area is underlain by generally loose, existing FILL, up to about 60 ft. thick. Based on experience with other projects in this area, it is our opinion that buildings constructed overtop of existing FILL and supported on shallow footings can experience substantial differential settlement. Several buildings we know of that were constructed on similar fills in this area have experienced about 8 to 12 inches of settlement. Such soils are susceptible to densification and volume reduction (hence settlement) as a result of vibrations, changed drainage conditions, and decay of buried organic matter. For the anticipated new building loads, we estimate that on the order of about 2 to 4 inches of potential foundation settlement would likely occur. Such settlements could adversely affect building performance and cause damage to the structure. For such site conditions, we believe that there are three main options for minimizing foundation settlement risks:

- In-place improvement of the relative density of the existing FILL so that foundation loads can be supported on conventional spread footing foundations without significant risk of detrimental settlement.

- Construction of deep foundations to support new loadings on competent native soil below the existing FILL.
- Complete or partial removal of the existing FILL and replacement with compacted Structural Fill, so that new foundation loads can be supported on conventional spread footing foundations.

In-place improvement of loose soil can be accomplished by procedures such as grouting, vibro-replacement, deep dynamic compaction (DDC), and preloading. Grouting fills the voids and increases relative density by welding the mass together. Vibro-replacement rearranges the existing particles and adds granular material to take up the volume loss. DDC rearranges particles by imparting a large amount of surface energy. Preloading simulates the weight of the proposed new building and forces settlements to occur before the actual building loads are applied.

In our opinion, this site is probably not suitable for grouting. In FILL soils, it is difficult to control where the grout penetrates. It is also difficult to predict grout quantities and, therefore, the costs are hard to control.

Vibro-replacement methods can be effective in medium to coarse grained fill soils, such as are present at this site. In this process of soil improvement, the loose, granular soils are rearranged into a denser configuration under the influence of a poker-type vibrator, usually accompanied by water jetting. The void created by rearrangement of the particles is filled with sand or gravel, which, under the action of the vibrator, is forced into the existing fill soils. The process is repeated on a grid pattern under the entire building footprint area.

Deep dynamic compaction is a method of improving and densifying soil by repeatedly dropping a heavy weight on a grid pattern from a large crane. In our opinion, this method would also probably be effective in improving the relative density of the existing FILL at this site. It was previously used successfully to improve loose existing FILL soils for the new WADOT Maintenance Building, approximately 3/4 of a mile southeast of the Faith Bible Church site. This method requires care to control flying debris and off-site vibrations which can be annoying and potentially damaging to neighboring buildings.

The preload method of soil improvement involves constructing a surcharge fill to simulate the weight of the new building and forcing settlement to occur before the actual building loads are applied. In granular soils, the induced settlements are relatively rapid. The method requires monitoring to measure the settlement that occurs during surcharge fill placement and rebound that occurs during surcharge removal. Preloading was used successfully to improve loose existing FILL soils for the new Group Health Riverfront Medical Center, approximately 1-1/2 miles south of the Faith Bible Church site.

Deep foundation support methods can include driven or auger-cast piling. Unless the loose surface subgrade is separately treated, such as with preloading or partial removal and replacement, however, the first floor of a pile-supported structure would also probably have to be structurally supported to minimize potential slab settlement.

Removing existing FILL and replacing it with compacted Structural Fill is an often used method for improving site foundation bearing conditions. At this site, however, because of the thickness of the existing FILL, the quantity of material that would have to be removed and replaced would probably make this method cost-prohibitive. Additionally, because of the local presence of a relatively large amount of debris, much of the existing fill would not be suitable for reuse as replacement Structural Fill; therefore, a considerable volume of import fill would be required.

Recently, on a similar project with similar soil conditions, we made a detailed cost comparison between:

a) Preloading,
b) Deep dynamic compaction, and
c) Driven steel H-piles, incorporating a structurally supported floor system.

Estimates for each method included the cost of additional detailed design recommendations and monitoring. The estimated cost for preloading was about 70 percent of the cost for deep dynamic compaction and about 35 percent of the cost for piles.

In our recent conversations, you indicated that shallow spread footing foundations on soil improved by preloading would be your preferred option for foundations and foundation soil improvement. The following site development and foundation design and construction recommendations are based on the preloading option.

General Site Preparation

Prior to site grading, it will be necessary to remove existing structures, debris, and vegetation. We recommend removing all existing buildings and their foundations, trees and major root structures, debris, and pavement. Additionally, all underground utilities should be removed from the building area. Following this, we recommend stripping the proposed building and paving areas of topsoil, surface vegetation and surficial root zones. We estimate that a 6 inch stripping depth should be sufficient in most areas. Selected topsoil materials could be stockpiled for later reuse in landscaping.

We recommend proof-rolling the paving areas with a heavy, vibratory compactor (10 ton minimum static weight), with at least six passes. This will help to densify the existing FILL subgrade and identify any loose areas that may have to be over-excavated and replaced with compacted Structural Fill.

Preloading of Building Area

Based on settlement analyses, it is our opinion that a surcharge of at least 1200 psf on the surface of the existing FILL will be required to simulate the proposed new building loads. In order to develop this amount of surcharge load, we estimate that an earth fill approximately 10 to 12 ft. high would be required, depending on the unit weight of the soil used. Compaction of the surcharge soil, although not necessary, would decrease the required surcharge height. Field density tests should be made to verify that the required total load is achieved.

Because of the generally granular nature of the existing FILL and the underlying native soil, we believe that the settlement induced by the surcharge will be relatively rapid. Based on the information currently available, we estimate settlement would probably be complete in less than about three to four months.

The surcharge should extend at full height a minimum of 10 ft. beyond the building perimeter. The sides of the surcharge should be sloped down at 1.5 H on 1 V to the surrounding existing grade (see Fig. 2). Before placing any surcharge fill, we recommend installing a series of about sixteen settlement plates on about a 60 to 70 ft. grid. We recommend that the plates be constructed in general accordance with the details shown on Fig. 12. Elevation measurements should be made:

- Immediately after installation.
- Upon completion of the surcharge fill.
- At one or two week intervals following completion, until measured settlements cease.
- After surcharge removal.

Settlement plate risers should be advanced as the surcharge is constructed and progressively removed as the surcharge is removed. Care should be taken by the Contractor to not disturb the settlement plates or risers during both surcharge construction and removal.

Settlement plate elevation data should be analyzed promptly to assess the completeness of the foundation soil settlement. When it is determined that settlement is complete, the surcharge can be removed. After surcharge removal, elevations of the settlement plates should be measured to determine the amount of foundation soil rebound. In our opinion, if less than about 3/4 inch of rebound occurs when the surcharge is removed, it will be possible to use conventional spread footings to support the new foundation loadings.

Foundations

In our opinion, the new structural loads could be supported on a system of conventional spread footing foundations constructed on existing FILL densified by preloading and proof-compacted before construction with a vibratory roller. We recommend that continuous or isolated spread footings be sized for an allowable net soil bearing pressure of 1500 psf. This bearing value could be increased by one-third for transient loading conditions, such as those resulting from wind or earthquake forces. This recommendation assumes that all exterior footings are embedded a minimum of 3 ft. below adjacent ground surface, for frost protection, and that interior footings are embedded a minimum of 1.5 ft. below the lowest adjacent slab grade. We recommend that continuous footings have a minimum width of 1.5 ft. Individual square footings should have a minimum dimension of 2.0 ft.

It is difficult at this time to estimate the potential settlement that might occur for a structure supported as recommended above. After analyzing the settlement and rebound data developed during the preloading, we will be better able to predict the foundation settlements. However, as a rule of thumb, foundation settlement can be expected to be similar to the amount of rebound that occurs upon removing the preload surcharge. Because of the granular nature of the foundations soils, we expect that, in general, foundation settlement will occur in an approximately elastic manner, almost as the loads are applied. Based on the low percentage of organic matter, less than about 2 percent, observed in the borings and test pits, we believe that long-term settlements will be negligible.

Lateral loadings from earth, wind, or seismic forces will be resisted by base friction and by passive earth pressure acting against the buried portions of the spread footings. In our opinion, passive earth pressures from compacted backfill against the sides of the footings could be estimated using an equivalent fluid pressure of 450H psf, where H is the depth below grade. This value should be used with a safety factor of about 1.5 and assumes that all backfill around the footings is compacted as Structural Fill and extends beyond the outside edge of the footing a minimum of two times the footing depth. For sliding friction at the base of footings, we recommend using a coefficient of friction between mass concrete and bearing soil of 0.45. This value should also be used with a safety factor of about 1.5.

The project site is located in seismic zone 2B, as identified by the 1993 Uniform Building Code. For structural design considerations, the seismic zone factor would be 0.20. The site coefficient would be 1.2.

Lateral Earth Pressures

For lateral design of below-grade walls that act as retaining walls, we recommend using an equivalent fluid pressure of 35H psf, where H is the height of the backfilled portion of the wall, in feet. This recommendation assumes that the wall is free to deflect when the backfill is placed (active earth pressure condition). We recommend using an equivalent fluid pressure of 55H psf if the top of the wall is totally restrained (at-rest pressure condition). Generally, if the deflection at the top of the wall can exceed about 0.001 times the free-standing wall height, it may be assumed unrestrained and the lower of the two earth pressures used. The earth pressures may be assumed to be distributed hydrostatically down the height of the wall. These earth pressures may also be used for design of temporary shoring, if necessary.

Excavations

Excavations up to about 5 ft. deep will be required for utilities, footings, and for general site grading. In our opinion, excavating can probably be accomplished with conventional equipment, such as backhoes, dozers, and rubber-tired loaders.

Since the maintenance of stable excavations is related to job safety, excavation stability should be the responsibility of the Contractor. All excavations should conform to Federal, State and local standards. Based on information from the borings and test pits, the site soils would classify as OSHA Type C, for excavation regulation purposes. For Type C soils, OSHA recommends that all unsupported, simple-slope excavations, 3 ft. deep or less, have a maximum allowable slope angle of 1.5 H on 1 V.

Subgrade Preparation

For areas where Structural Fill will be placed for the support of the building or pavements, we recommend scarifying the proof-rolled surface to a depth of about 6 ins., conditioning the soil with moisture, if necessary, and mixing with about 6 ins. of Structural Fill material. The surface should then be compacted with several passes of the compaction equipment to obtain a minimum density of 95 percent of the Modified Proctor maximum dry density (AASHTO T-180), followed by placing and compacting subsequent lifts of Structural Fill.

New footing subgrade surfaces should be proof-compacted with a vibratory roller to achieve a density of not less than 95 percent of the Modified Proctor density.

Structural Fill

We recommend that Structural Fill material consist of clean, reasonably well graded sand and gravel, having a maximum size of about 6 ins. and not more than about 15 percent by weight passing the No. 200 sieve. That portion passing the No. 200 sieve should be non-plastic. Using these criteria, most of the existing FILL and native gravelly SAND would be reusable as compacted Structural Fill, as long as over-sized (+6 ins.) fragments are first removed. Some of the existing FILL will not be reusable because of its high debris content. Samples of imported fill material should be provided for approval by the soils engineer.

Structural Fill should be brought to optimum moisture content, placed in thin lifts (not more than 10 ins. in loose thickness), and compacted to a density of not less than 95 percent of the Modified Proctor maximum dry density (AASHTO T-180). Laboratory maximum density testing should be performed on all potential Structural Fill materials to establish moisture and density criteria before fill placement begins. We recommend that Structural Fill placement and compaction be continuously monitored by a an experienced soils engineer or engineering technician, representing the Owner.

Floor Slabs

In our opinion, compacted Structural Fill, or existing FILL soil densified by preloading, will provide sufficient support for slabs-on-grade. Slabs should be designed using a standard modulus of subgrade reaction, k, of about 200 pci for native or fill soil subgrade, compacted to 95 percent of the Modified Proctor maximum density.

We recommend that the subgrade under proposed slab areas be compacted and then capped with a minimum 4 inch thick cushion of 3/4 inch minus crushed rock or gravel to provide uniform support. We recommend that laboratory compaction testing be performed prior to field work on representative samples of materials proposed for slab cushion, as well as subgrade.

Pavements

We estimate that the R-value for the existing FILL as pavement subgrade would be about 70. The following recommended pavement design is based on this value, a 20 year design life, and the following assumed traffic indexes:

- Automobile parking areas - TI = 4.0.
- Driveways - TI = 5.0.

For parking area pavements, we recommend using a minimum of 2 ins. of asphalt surfacing, overlying 4 ins. of crushed rock base course. For driveways, we recommend increasing the crushed rock base course thickness to 6 ins. The base course should be placed over proof-compacted existing FILL, or compacted Structural Fill. We recommend against paving over topsoil, loose existing FILL, or other loose, soft, or organic soils.

The crushed rock base course should meet the gradation requirements described in the 1994 Standard Specifications For Road, Bridge, and Municipal Construction, WDOT Section 9-03.9 (3), as follows:

Sieve No.	Percent by Weight Passing
1-1/4 in.	100
5/8 in.	50-80
1/4 in.	30-50
No. 40	3-18
No. 200	<7.5
Minimum % Fracture	75
Minimum Sand Equivalent	40

The crushed rock base course should be placed in a single lift and compacted to a density of not less than 95 percent of the maximum, as determined by the Modified Proctor Method (AASHTO T-180). We recommend that the existing FILL or Structural Fill making up the subgrade beneath the base course also be compacted to 95 percent.

The asphalt concrete pavement should meet the specifications for Class B Asphalt Concrete, as described in the Standard Specifications for Road, Bridge, and Municipal Construction, WADOT. We recommend that it be compacted to a minimum of 92 percent of the theoretical maximum, or Rice density, as determined by WADOT Test Method 705.

Site Drainage and Erosion Control

Runoff from roof drains and pavements should be collected and discharged into storm sewers, or into appropriately designed stormwater detention facilities. This site is located over the Spokane/Rathdrum Regional Aquifer. Therefore, plans for development are required to include special provisions for handling runoff which may contain constituents that could degrade aquifer water quality. Typically, this involves a grassy swale for detention of stormwater accumulation and a drywell for overflow disposal.

In our opinion, the permeability of the in-place existing FILL and native gravelly sand at the probable elevation of drywells on this site would probably be on the order of about 10 to 100 ft./day. We recommend that if drywells are used for disposal, they be located at least 30 ft. away from the new building. Final grading of the site should be designed to promote drainage away from the building.

For erosion protection, permanent unpaved slopes should be constructed no steeper than 2 H on 1 V. All newly constructed slopes should be covered with sod, or seeded during the first growing season after construction.

LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on our interpretation of subsurface conditions and assume that the information obtained from the borings and test pits is representative of subsurface conditions throughout the site. If, during construction, subsurface conditions different from those in the borings and test pits and described herein appear to be present beneath the site, we should be advised at once, so that we can review these conditions and reconsider our recommendations where necessary.

This report was prepared for the use of Faith Bible Church for the design of their new building, in Spokane, Washington. It should be made available to potential contractors and/or the Contractor for information on factual data only; that is, borings, test pits, logs, and soil samples. This report should not be used for contractual purposes, or as a warranty of interpreted subsurface conditions, such as those indicated by the boring or test pit logs or discussions of subsurface conditions contained herein.

We recommend that close quality control be exercised during Structural Fill placement and compaction and in the preparation of the foundations. We recommend that all earthwork tasks, including preload surcharge construction, be monitored and observed by a geotechnical engineer or engineering technician.

If there is substantial lapse of time between the submission of this report and the start of the work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, we recommend that this report be reviewed to determine that the conclusions and recommendations contained herein are still applicable. If you desire, we will review those portions of the plans and specifications which pertain to earthwork and foundations to determine if they are consistent with our recommendations. We recommend that you retain us to monitor preloading, analyze preload settlement data, observe site earthwork, foundation construction, and other foundation related field operations as may be necessary.

This report does not include the review of historical data, reconnaissance, or testing to assess the presence of any hazardous substances.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely making test borings, test pits, or other explorations. Such unexpected conditions frequently require that additional expenditures be made to obtain a properly constructed project. We recommend establishing a contingency fund to accommodate such unexpected conditions.

Sincerely,

GIFFORD CONSULTANTS, INC.

GRANT R. CUMMINGS
STATE OF WASHINGTON
0027912
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 6/6/95
10/6/94

Grant R. Cummings, P.E., P.G.
Geological Engineer

ALLEN B. GIFFORD
STATE OF WASHINGTON
14609
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 05/23/96
Oct 16, 1994

Allen B. Gifford
President

Encl: Important Information About Your Geotechnical Engineering Report
Figs. 1 through 12
Table 1

Important Information About Your Geotechnical Engineering Report

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON PROJECT-SPECIFIC FACTORS

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

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

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

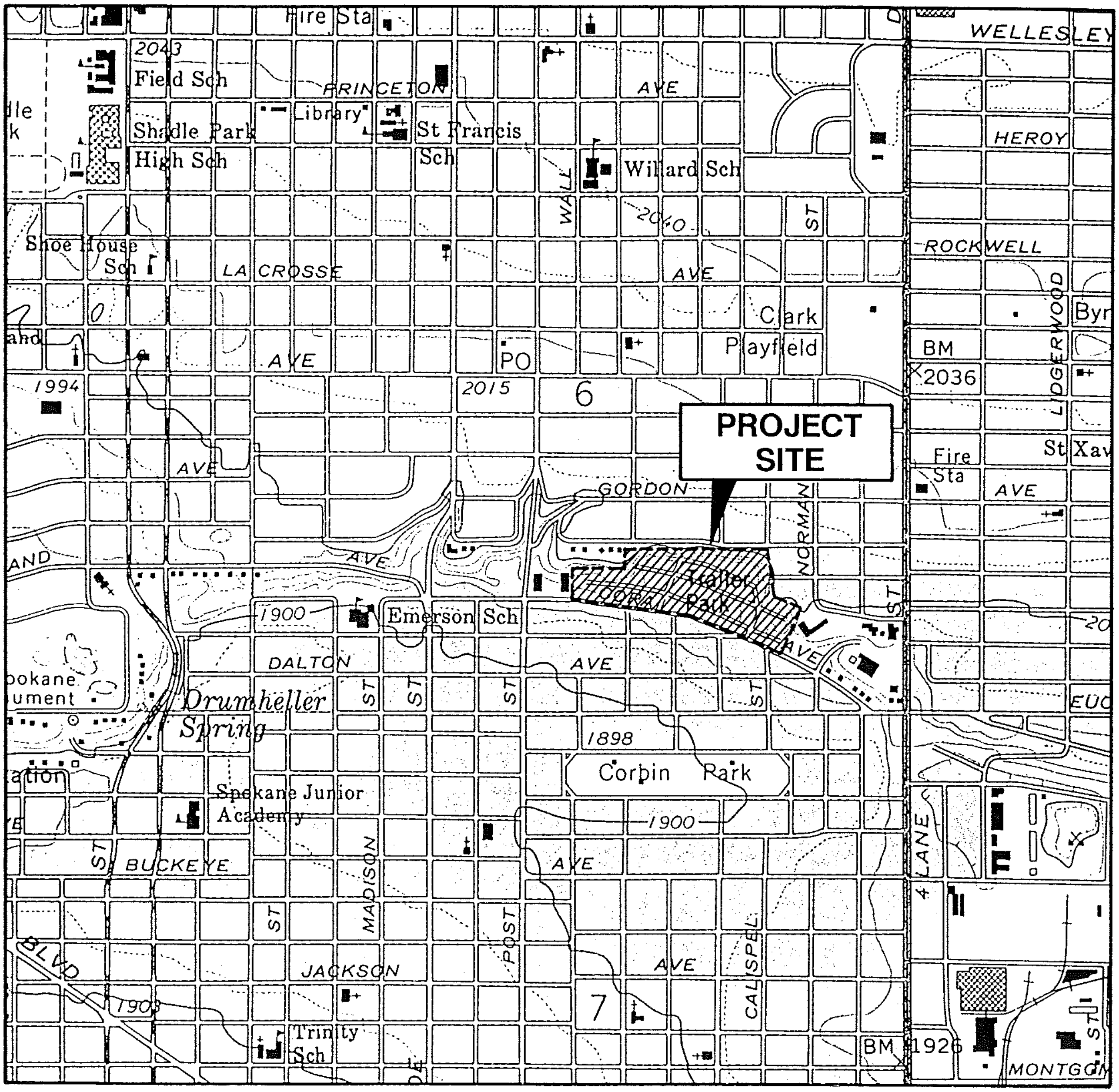
Geotechnical engineers cannot accept responsibility for problems which may develop if they are not consulted after factors considered in their reports have changed.

MOST GEOTECHNICAL "FINDINGS" ARE PROFESSIONAL ESTIMATES.

Site exploration identifies subsurface conditions only at those points where samples are taken and when they are taken, but the physical means of obtaining subsurface data precludes the determination of precise conditions. Consequently, the information obtained is intended to be sufficiently accurate for design, but is subject to interpretation. Additionally, data derived through sampling and subsequent laboratory testing are extrapolated by the geotechnical engineer who then renders an opinion about overall subsurface conditions, their likely reaction to proposed construction activity, and appropriate foundation design. Even under optimal circumstances actual conditions may differ from those opined to exist, because no geotechnical engineer, no matter how qualified, and no subsurface exploration program, no matter how comprehensive, can reveal what is hidden by earth, rock, and time. For example, the actual interface between materials may be far more gradual or abrupt than the report indicates, and actual conditions in areas not sampled may differ from predictions. Nothing can be done to prevent the unanticipated, but steps can be taken to help minimize their impact. For this reason, most experienced owners retain their geotechnical consultant through the construction stage, to identify variances, conduct additional tests which may be needed, and to recommend solutions to problems encountered on site. Prudent owners establish contingencies to accommodate such variations in subsurface conditions as exposed during construction.

SUBSURFACE CONDITIONS CAN CHANGE

Subsurface conditions may be modified by constantly changing natural forces. Because a geotechnical engineering report is based on conditions which existed at the time of subsurface exploration, construction decisions should not be based on a geotechnical engineering report whose adequacy may have been affected by time. Speak with the geotechnical consultant to learn if additional tests are advisable before construction starts. For example, groundwater conditions commonly vary seasonally. Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater



Spokane, Wash.

Sec. 6, T25N, R43E W.M.

N

0 1000 2000

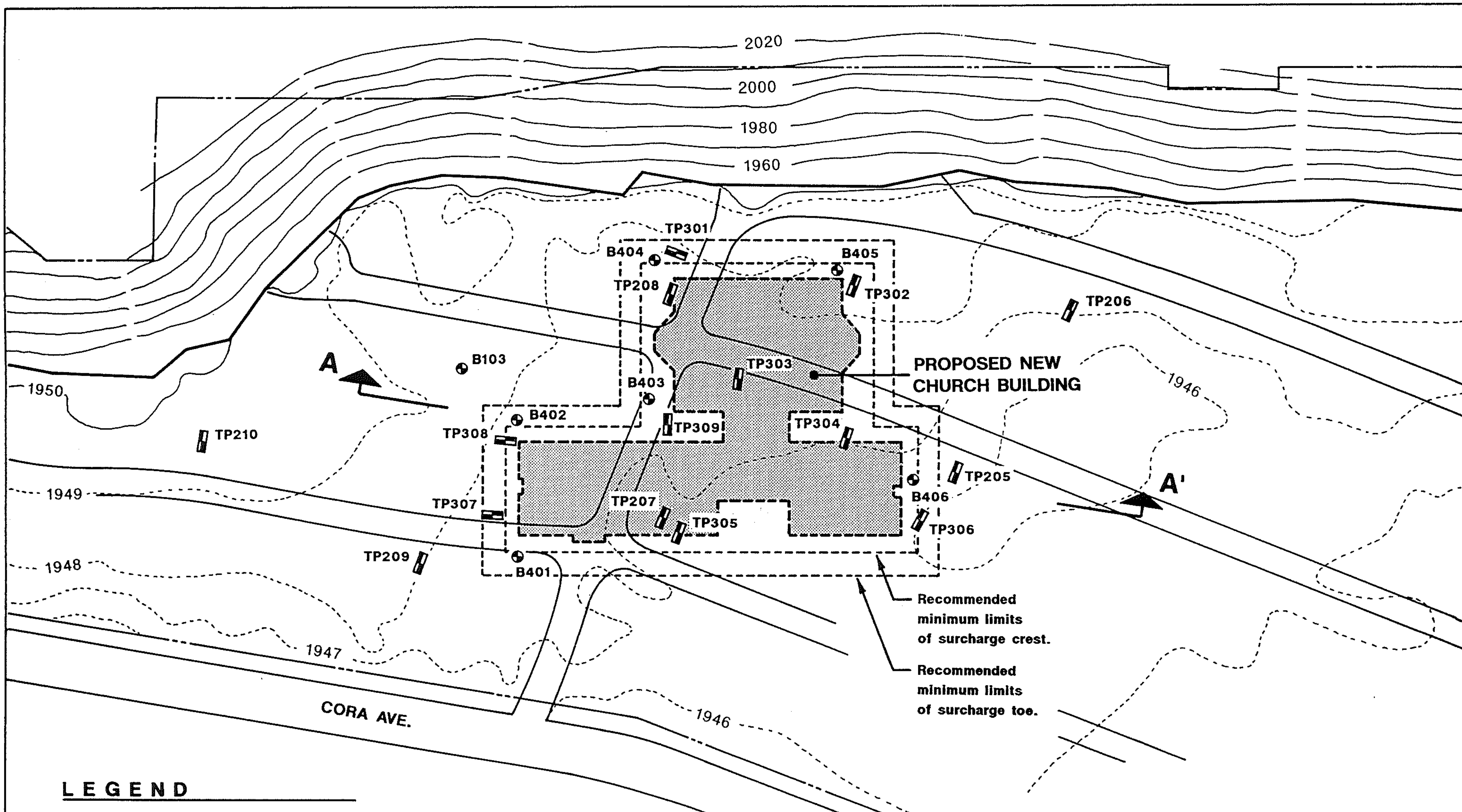
SCALE IN FEET

Base map from USGS 7.5 min. quadrangle titled "Spokane NW, Wash.", 1974 photorevised 1986.

FAITH BIBLE CHURCH Spokane, Wash.	
VICINITY MAP	
OCT. 1994	E-1487-02
GCI GIFFORD CONSULTANTS, INC. E. 11616 MONTGOMERY #4 SPOKANE, WA 99206 GEOTECHNICAL ENGINEERING	**FIG. 1**

Drawn By TOB Drwg. Date 10-4-94 Checked By Field Rep. Revision date

Drawing Date 9.30.94 Drawn By TDB Checked By [illegible] Field Rep. ______



LEGEND

B401 ⊕ 1994 Boring Location
B103 ⊕ 1991 Boring Location
TP301 1994 Test Pit Location
TP205 1991 Test Pit Location
------ 1 ft. contour
—— 10 ft. contour
Subsurface Profile Location

SCALE: 1"=100'

FIG. 2

FAITH BIBLE CHURCH
Spokane, Wash.

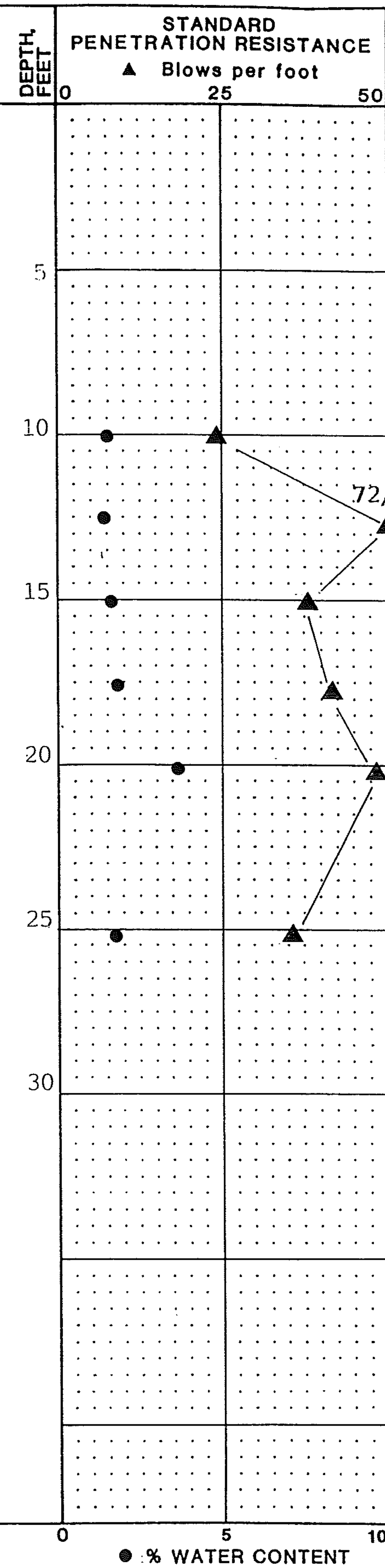
SUBSURFACE EXPLORATION PLAN

OCTOBER 1994 E-1487-02

GCI GIFFORD CONSULTANTS, INC.
GEOTECHNICAL ENGINEERS
2020 E. Springfield Ave., Spokane, WA 99202

FIG. 2

SOIL DESCRIPTION Surface Elevation: 1947.5'	DEPTH, FEET	SAMPLES	GROUND WATER
Medium to very dense, brown, gravelly coarse SAND (FILL); moist; small pieces of glass and plastic noted in cuttings from 0-14 ft.		1 2	No Ground Water Encountered During Drilling
Dense, brown, gravelly SAND (NATIVE) with trace of silt; moist.	14.5	3 4 5 6	
Bottom of Boring Completed 08/24/94	25.5		



LEGEND

I 2" O.D. Split-Spoon Sample
II 3" O.D. Thin-Wall Sample
* Sample Not Recovered
Impervious Seal
Water Level
Piezometer Tip
Plastic Limit —|—●—|— Liquid Limit

NOTE: The stratification lines represent the approximate boundaries between soil types. The actual transitions may be gradual.

Drawn by TDB Drwg. date 10.3.94 Checked by SRC Field rep. RWB Revision date

Faith Bible Church
Spokane, Washington

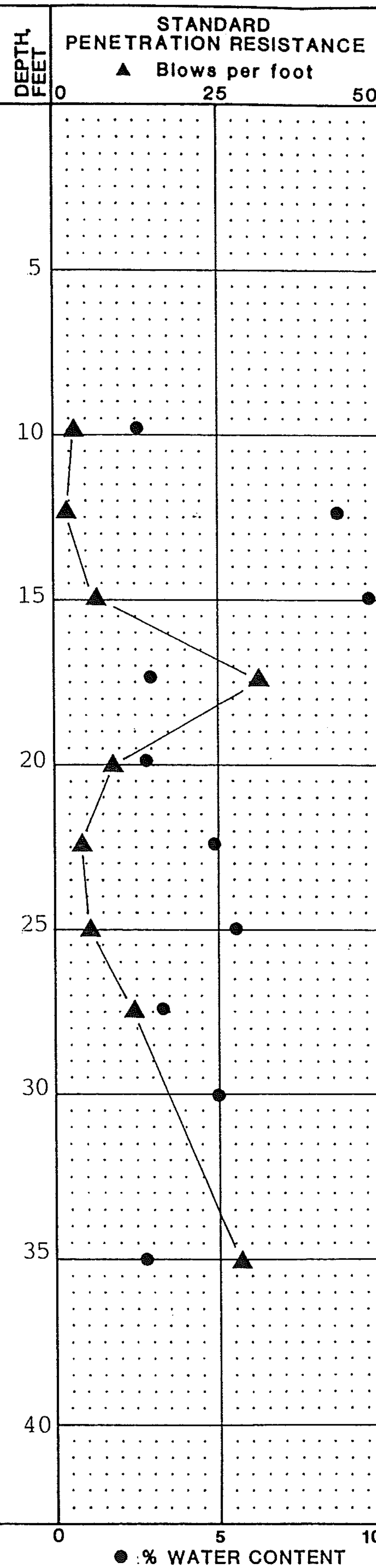
LOG OF BORING NO. 401

August 1994 E-1487-02

GIFFORD CONSULTANTS, INC.
Geotechnical Engineers

FIG. 3

SOIL DESCRIPTION Surface Elevation: 1948'	DEPTH, FEET	SAMPLES	GROUND WATER
Very loose, brown, coarse SAND (FILL) with trace of gravel; moist.	10.5	1	No Ground Water Encountered During Drilling
Very loose to loose, brown, sandy GRAVEL (FILL); with trace of silt, asphalt fragments, and occasional cobbles; moist.	21.0	2, 3, 4, 5	
Loose, brown, SAND (FILL); trace of gravel, trace of silt; moist.	27.5	6, 7, 8	
Medium dense, brown to gray, gravelly SAND (NATIVE); moist.	35.5	9, 10	
Bottom of Boring Completed 08/24/94			



LEGEND

I 2" O.D. Split-Spoon Sample
II 3" O.D. Thin-Wall Sample
* Sample Not Recovered
Plastic Limit —●— Liquid Limit
Impervious Seal
Water Level
Piezometer Tip

NOTE: The stratification lines represent the approximate boundaries between soil types. The actual transitions may be gradual.

Faith Bible Church
Spokane, Washington

LOG OF BORING NO. 402

August 1994 E-1487-02

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FIG. 4

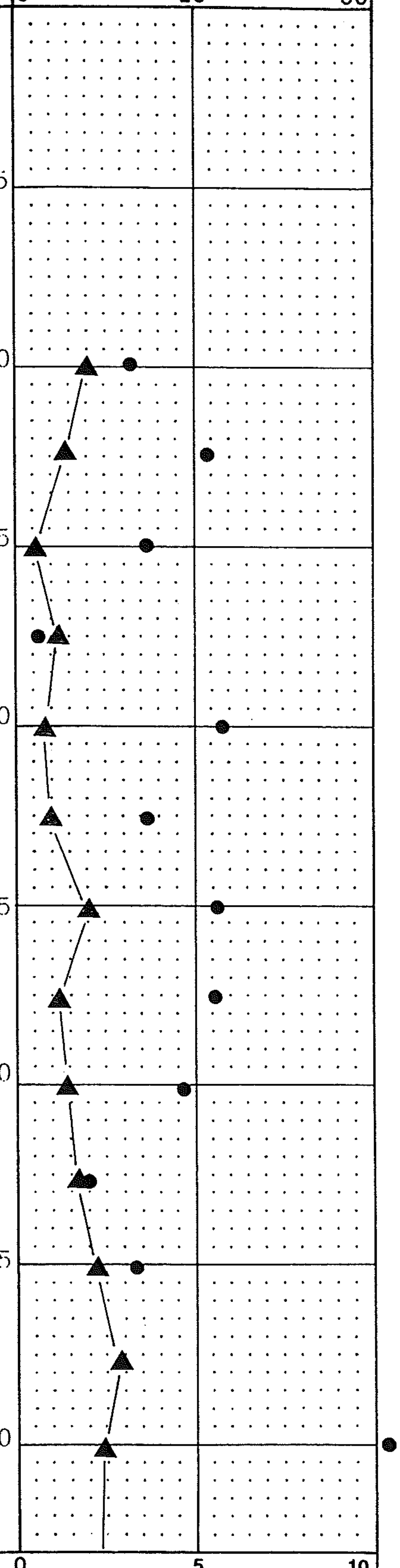
Drawn by TDD Drwg. date 10.3.94 Checked by BAC Field rep. B.W.B. Revision date

SOIL DESCRIPTION Surface Elevation: 1947.5'	DEPTH, FEET	SAMPLES	GROUND WATER	DEPTH, FEET
Loose to very loose, brown, gravelly SAND (FILL) with trace of silt; moist, glass fragments noted in cuttings at 29 ft.		1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13	No Ground Water Encountered During Drilling	5, 10, 15, 20, 25, 30, 35, 40

continued

STANDARD PENETRATION RESISTANCE

▲ Blows per foot



● % WATER CONTENT

LEGEND

I 2" O.D. Split-Spoon Sample

II 3" O.D. Thin-Wall Sample

* Sample Not Recovered

Impervious Seal

Water Level

Piezometer Tip

Plastic Limit —●— Liquid Limit

NOTE: The stratification lines represent the approximate boundaries between soil types. The actual transitions may be gradual.

Faith Bible Church
Spokane, Washington

LOG OF BORING NO. 403

August 1994 E-1487-02

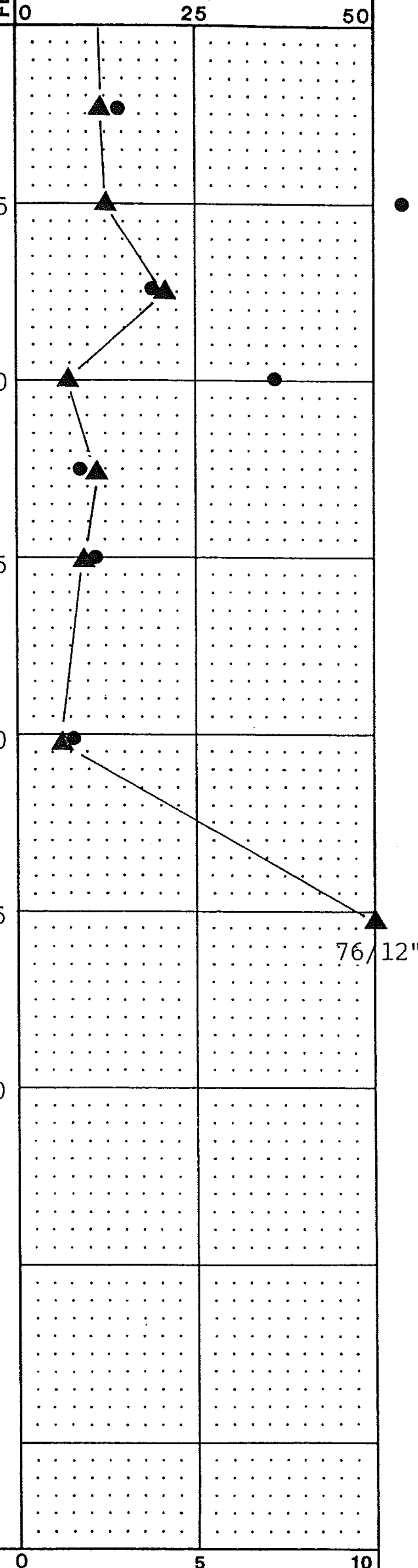
GIFFORD CONSULTANTS, INC.
Geotechnical Engineers

FIG. 5a

Drawn by TPB Drwg. date 10-3-94 Checked by [signature] Field rep. B.W.B. Revision date

SOIL DESCRIPTION Surface Elevation: 1947.5'	DEPTH, FEET	SAMPLES	GROUND WATER	DEPTH, FEET
continued			No Ground Water Encountered During Drilling	
Medium dense, brown to gray brown, gravelly SAND (FILL), with trace silt; moist, porcelain fragments at 40 ft.	41.0	14 15 16 17 18 19 20		45 50 55 60
Very dense soil, no sample recovery, possibly native.	62.0	*21		65
Bottom of Boring Completed 8/25/94	65.5			70

STANDARD PENETRATION RESISTANCE



LEGEND

- I 2" O.D. Split-Spoon Sample
- II 3" O.D. Thin-Wall Sample
- * Sample Not Recovered
- Impervious Seal
- Water Level
- Piezometer Tip
- Plastic Limit —●— Liquid Limit

NOTE: The stratification lines represent the approximate boundaries between soil types. The actual transitions may be gradual.

Faith Bible Church
Spokane, Washington

LOG OF BORING NO. 403

August 1994 E-1487-02

GIFFORD CONSULTANTS, INC.
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FIG. 5b

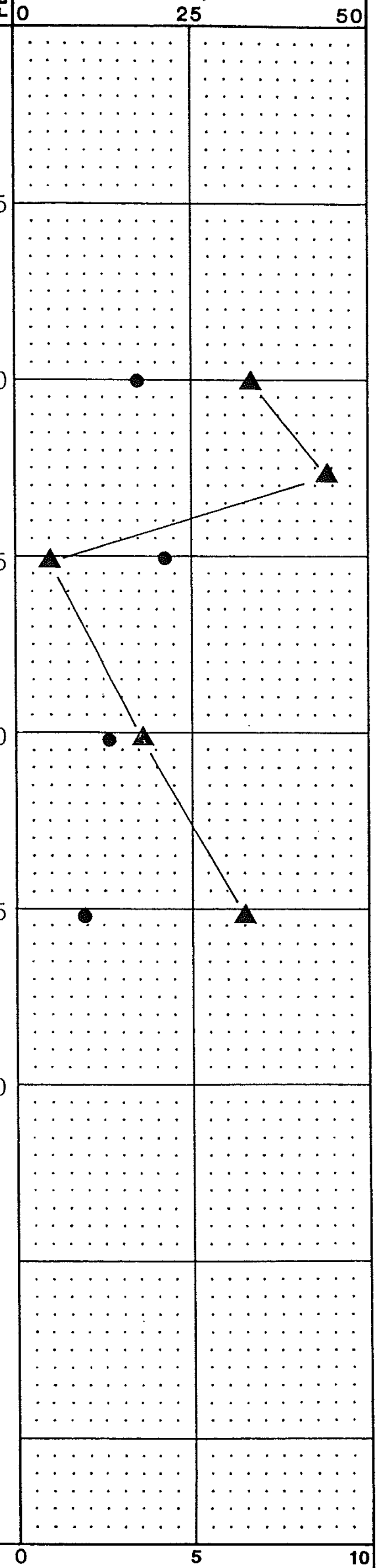
Drawn by TDB Drwg. date 10-3-94 Checked by [illegible] Field rep. B.W.B Revision date

SOIL DESCRIPTION Surface Elevation: 1948'	DEPTH, FEET	SAMPLES	GROUND WATER
Loose to dense, brown, gravelly SAND (FILL) with trace of silt, occasional glass fragments; moist.		1 *2 3	No Ground Water Encountered During Drilling
Medium dense to dense, grayish brown, gravelly SAND (NATIVE); moist.	19.5	4 5	
Bottom of Boring Completed 8/26/94	25.5		

DEPTH, FEET

STANDARD PENETRATION RESISTANCE

▲ Blows per foot



LEGEND

I 2" O.D. Split-Spoon Sample

II 3" O.D. Thin-Wall Sample

* Sample Not Recovered

Impervious Seal

Water Level

Piezometer Tip

Plastic Limit —|—●—|— Liquid Limit

NOTE: The stratification lines represent the approximate boundaries between soil types. The actual transitions may be gradual.

Faith Bible Church
Spokane, Washington

LOG OF BORING NO. 404

August 1994 E-1487-02

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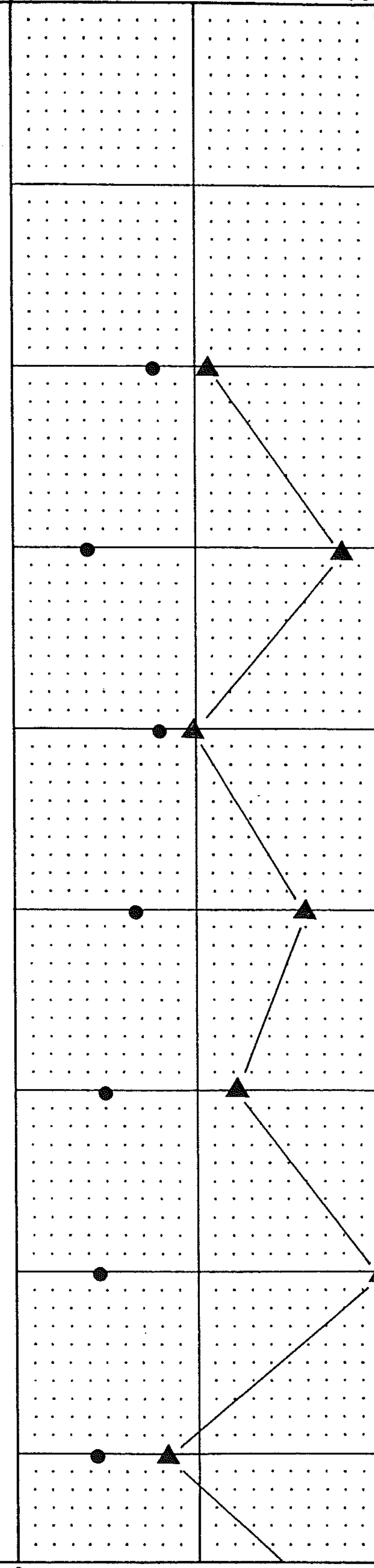
FIG. 6

Drawn by TDB Drwg. date 10-7-94 Checked by [signature] Field rep. BWB Revision date

SOIL DESCRIPTION Surface Elevation: 1948'	DEPTH, FEET	SAMPLES	GROUND WATER	DEPTH, FEET	STANDARD PENETRATION RESISTANCE ▲ Blows per foot 0 25 50
Medium dense to dense, brown to gray, slightly silty, gravelly SAND (FILL), with occasional glass, china, and tire fragments; moist.		1 I 2 I 3 I 4 I 5 I 6 I 7 I	No Ground Water Encountered During Drilling	5 10 15 20 25 30 35 40	
continued					0 5 10 ● % WATER CONTENT

LEGEND

I 2" O.D. Split-Spoon Sample
II 3" O.D. Thin-Wall Sample
* Sample Not Recovered
Plastic Limit —⊢●⊣— Liquid Limit
Impervious Seal
Water Level
Piezometer Tip

NOTE: The stratification lines represent the approximate boundaries between soil types. The actual transitions may be gradual.

Faith Bible Church
Spokane, Washington

LOG OF BORING NO. 405

August 1994 E-1487-02

GIFFORD CONSULTANTS, INC.
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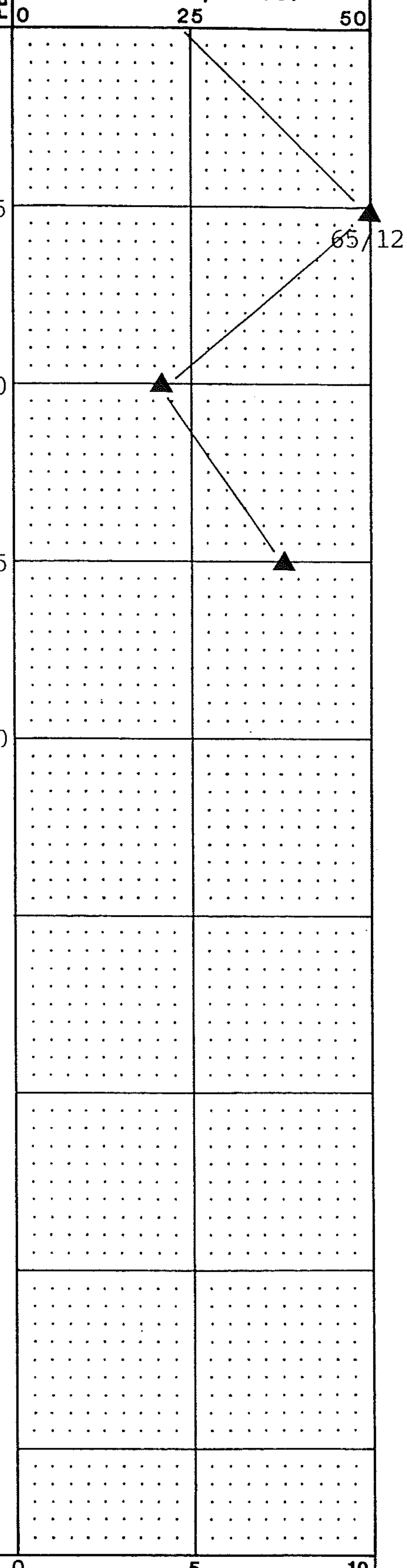
FIG.7a

Drawn by TDB Drwg. date 10-3-94 Checked by [signature] Field rep. B.W.B. Revision date

SOIL DESCRIPTION Surface Elevation: 1948'	DEPTH, FEET	SAMPLES	GROUND WATER	DEPTH, FEET
Continued Medium dense to dense, brown to gray, slightly silty, gravelly SAND (FILL), with occasional glass, china, and tire fragments; moist.		*8 I *9 I *10 I	No Ground Water Encountered During Drilling	45 50 55
Bottom of Boriing Completed 8/26/94	55.5			60

STANDARD PENETRATION RESISTANCE

▲ Blows per foot



LEGEND

I 2" O.D. Split-Spoon Sample

II 3" O.D. Thin-Wall Sample

* Sample Not Recovered

Impervious Seal

Water Level

Piezometer Tip

Plastic Limit —|—●—|— Liquid Limit

NOTE: The stratification lines represent the approximate boundaries between soil types. The actual transitions may be gradual.

Faith Bible Church
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LOG OF BORING NO. 405

August 1994 E-1487-02

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FIG. 7b

Drawn by ___ Drwg. date 10.3.94 Checked by ___ Field rep. BWB Revision date ___

SOIL DESCRIPTION Surface Elevation: 1946'	DEPTH, FEET
Very loose to medium dense, brown, slightly silty, gravelly <u>SAND</u> (FILL); moist.	
Dense soil, no sample recovery, possibly native.	30.5
Bottom of Boring Completed 8/29/94	33.0

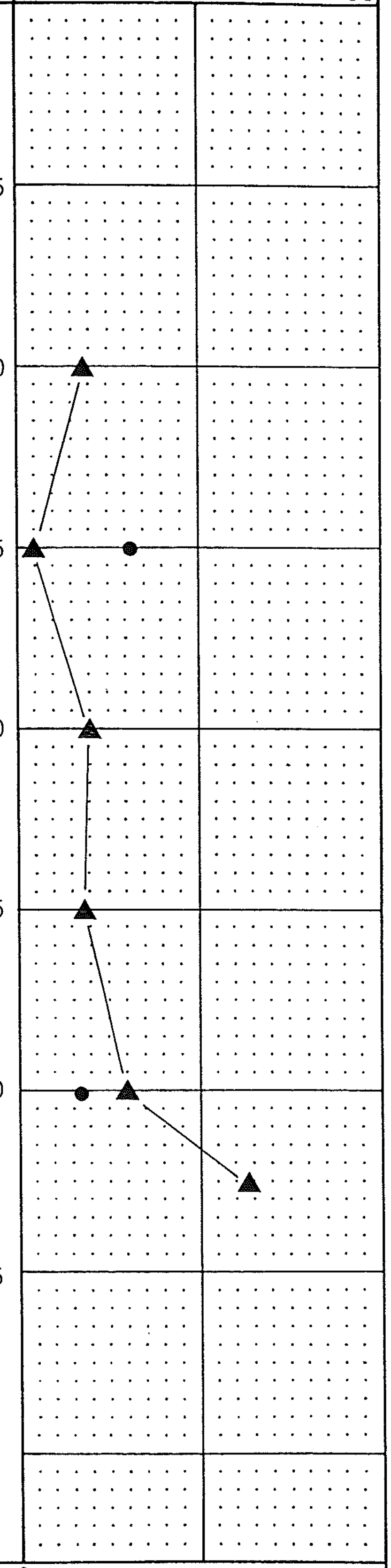
SAMPLES: *1, 2, *3, *4, 5, *6

GROUND WATER: No Ground Water Encountered During Drilling

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

STANDARD PENETRATION RESISTANCE

▲ Blows per foot



● % WATER CONTENT

LEGEND

I 2" O.D. Split-Spoon Sample

II 3" O.D. Thin-Wall Sample

* Sample Not Recovered

Impervious Seal

Water Level

Piezometer Tip

Plastic Limit —●— Liquid Limit

NOTE: The stratification lines represent the approximate boundaries between soil types. The actual transitions may be gradual.

Faith Bible Church
Spokane, Washington

LOG OF BORING NO. 406

August 1994 E-1427-02

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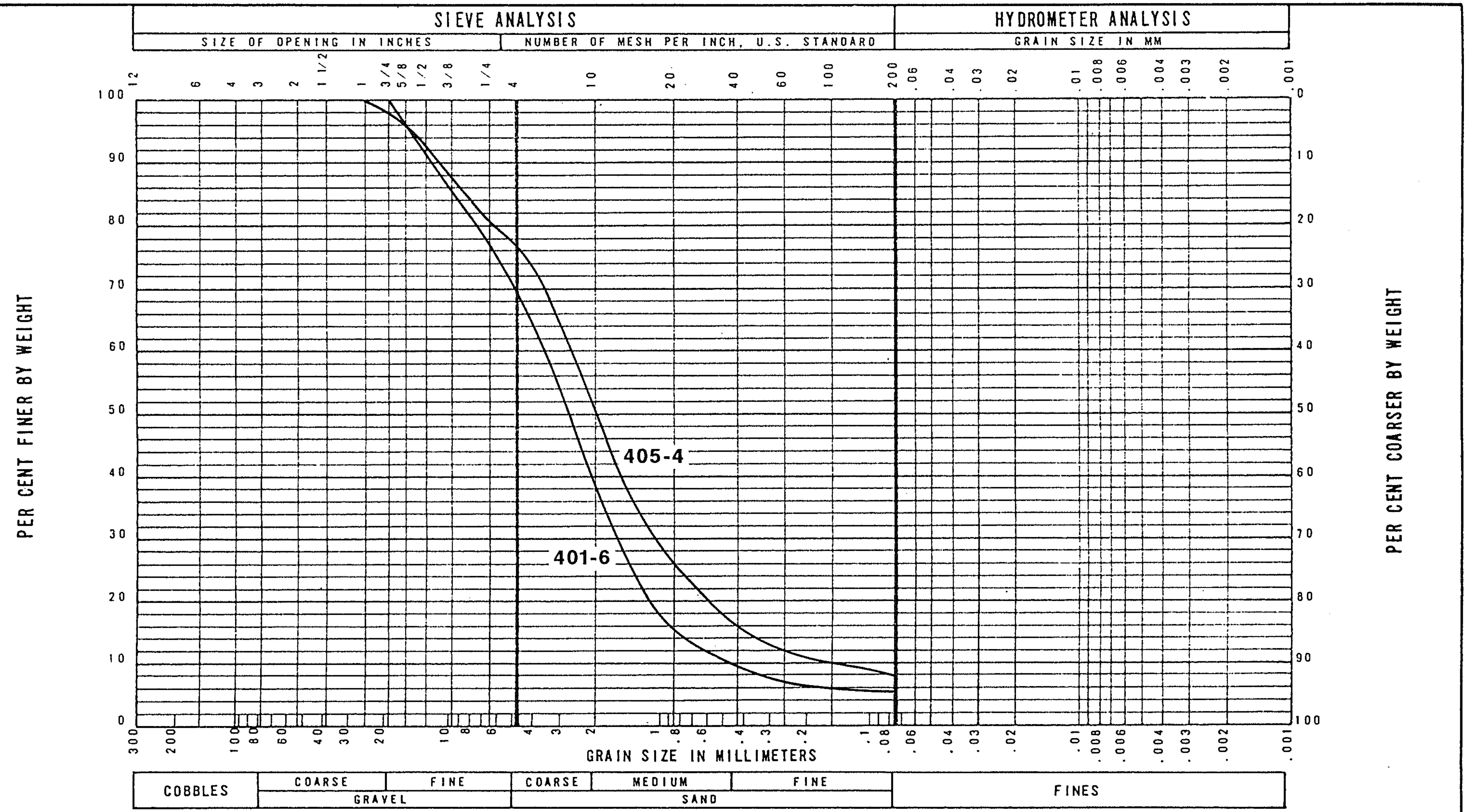
Geotechnical Engineers

FIG. 8

Drawn by TDB Drwg. date 10-3-94 Checked by [signature] Field rep. B.W.B. Revision date

Drawn By______ Drwg. Date______ Checked By SRC Field Rep.______



SAMPLE NO.	DEPTH-FT.	U.S.C.	CLASSIFICATION	NAT. W.C. %	LL	PL	PI
401-6	24.0-25.5	SW	Dark brown, gravelly SAND; trace of silt.	1.6	-	-	-
405-4	24.0-25.5	SP	Brown, slightly silty, gravelly SAND	3.4	-	-	-

Faith Bible Church
Spokane, Washington

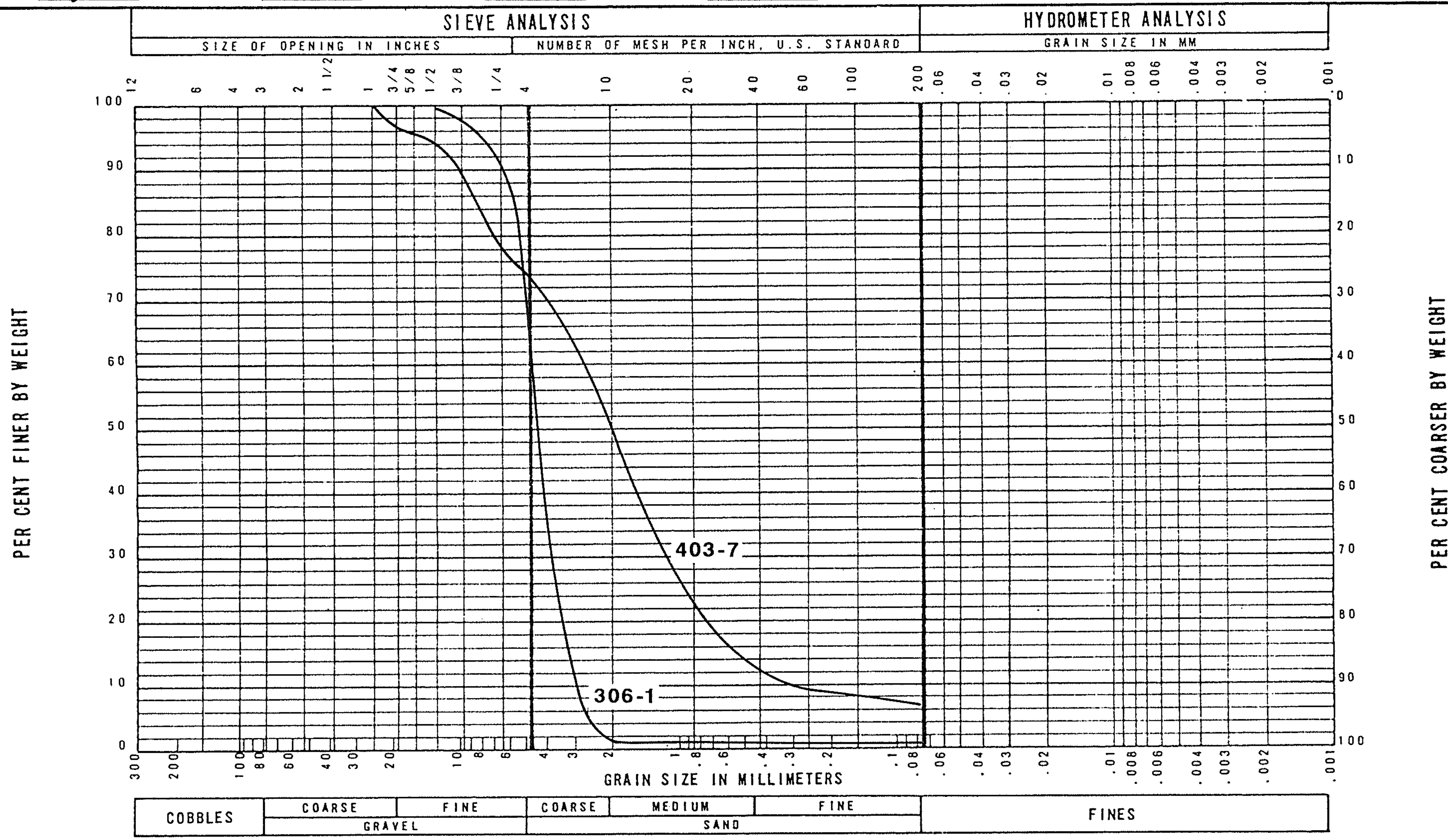
GRAIN SIZE CLASSIFICATION

September 1994 E-1487-02

GIFFORD CONSULTANTS, INC.
Geotechnical Engineers

FIG. 9

Drawn By TDD Drwg. Date 10-3-94 Checked By [illegible] Field Rep.



SAMPLE NO.	DEPTH-FT.	U.S.C.	CLASSIFICATION	NAT. W.C. %	LL	PL	PI
306-1	8.0	SP	Gray, slightly gravelly, coarse SAND	0.3	-	-	-
403-7	24.0-25.5	SW	Brown, slightly silty, gravelly SAND	5.5	-	-	-

Faith Bible Church
Spokane, Washington

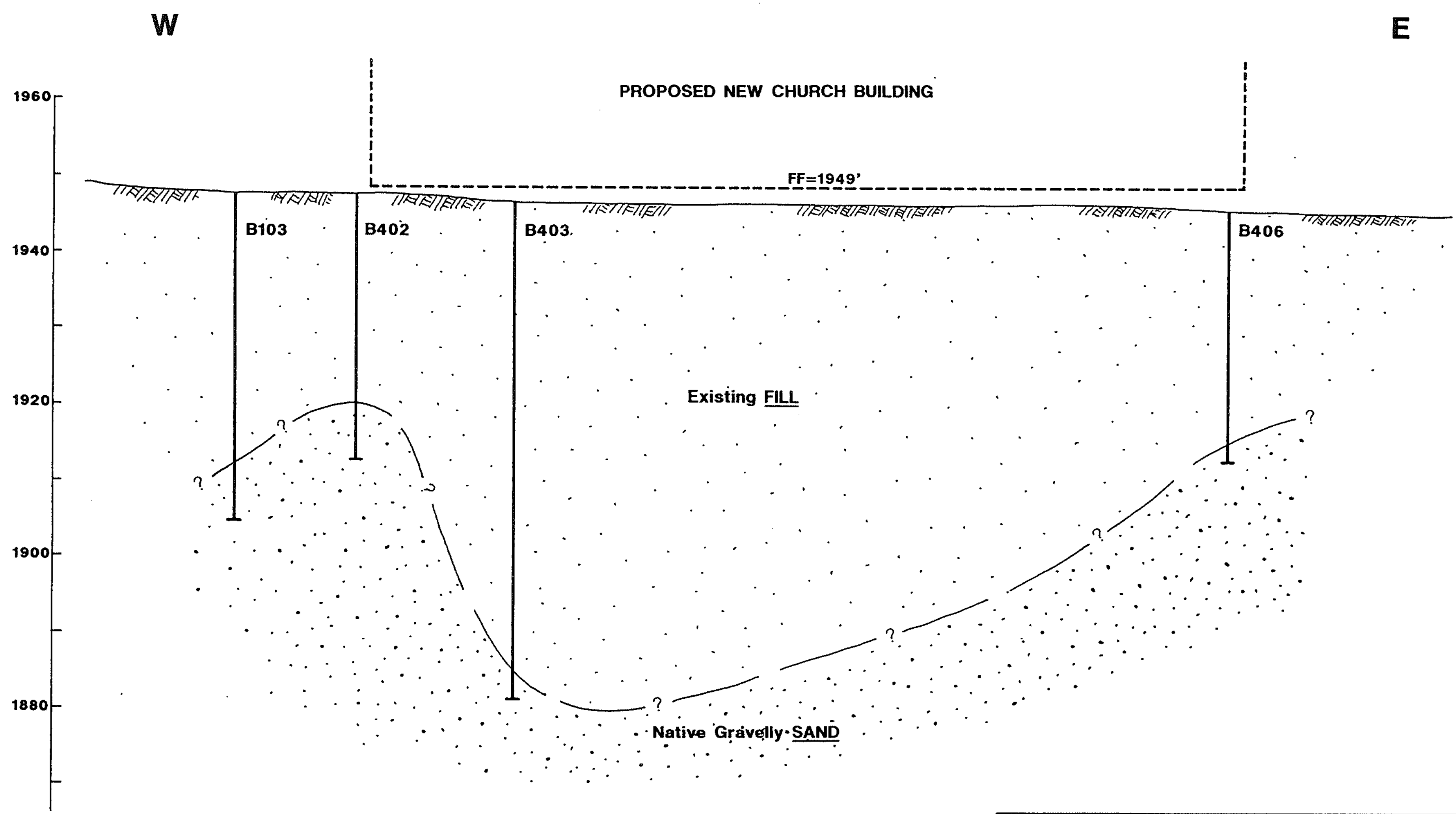
GRAIN SIZE CLASSIFICATION

September 1994 E-1487-02

GIFFORD CONSULTANTS, INC.
Geotechnical Engineers

FIG. 10

Drawing Date 10·3·94 Drawn By TDB Checked By SRC Field Rep. ________



SCALE: 1"=50' H
1"=20' V

FAITH BIBLE CHURCH
Spokane, Wash.

SUBSURFACE SECTION A-A'

OCT. 1994 E-1487-02

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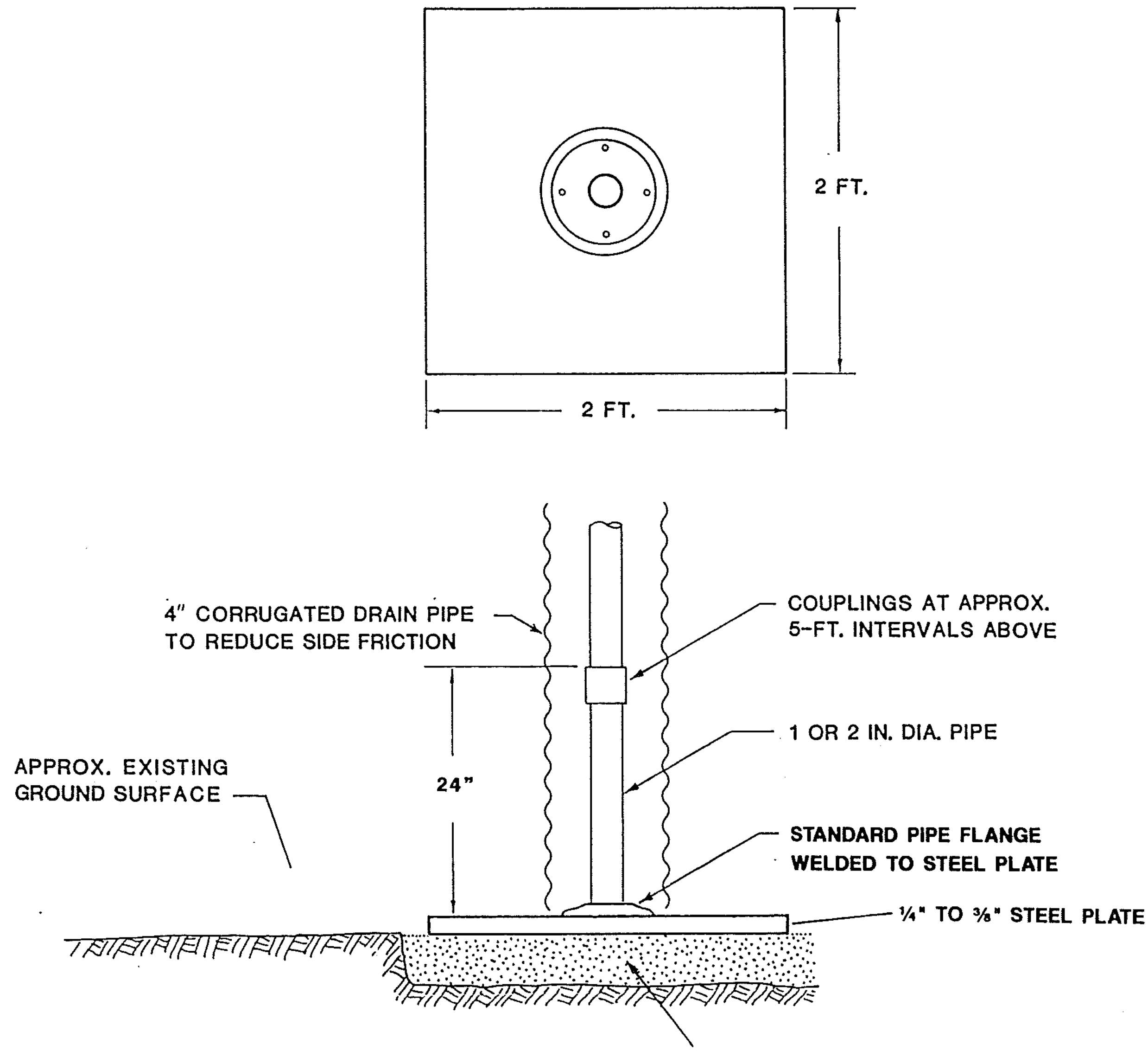
GEOTECHNICAL ENGINEERS
2020 E. Springfield Ave., Spokane, WA 99202

FIG. 11

FIG. 11



NOT TO SCALE

FAITH BIBLE CHURCH
Spokane, Wash.

STANDARD STEEL SETTLEMENT PLATE DETAILS

OCT. 1994 | E-1487-02

GCI GIFFORD CONSULTANTS, INC.
GEOTECHNICAL ENGINEERING

FIG. 12

Drawn By ___ Drwg. Date ___ Checked By ___ Field Rep. ___ Revision date ___

TABLE 1
TEST PIT LOGS

TP-301	GSE* 1948'	July 13, 1994
0-15.0'	Loose, brown-gray, gravelly <u>SAND</u> (FILL), with trace of silt and occasional rounded cobbles, boulders and debris, including rebar, metal scraps, rubber, glass, brick, and concrete fragments; moist. At 5.5 ft., could probe 0.5 to 1.0 ft. with 1/2 in. diameter steel rod. Excavation caved easily from 3 to 7 ft. and 9 to 15 ft. Silty sand lenses noted between 7 and 8 ft. deep. No ground water encountered *GSE = Ground surface elevation	

TP-302	GSE 1948'	July 13, 1994
0-14.0'	Loose to medium dense, brown-gray, slightly silty, gravelly <u>SAND</u> (FILL), with occasional rounded cobbles and boulders and debris consisting of concrete, metal scraps, glass and brick fragments; dry. At 5.0 ft. below ground surface, could probe 3 to 4 ins. with 1/2 in. diameter steel rod. Some caving occurred below 8 ft. Silty sand lenses mixed with debris noted at approx. 7 ft. No ground water encountered	

TP-303	GSE 1947.5'	July 13, 1994
0-9.5'	Loose to medium dense, brown-gray, slightly silty, gravelly <u>SAND</u> (FILL), with occasional rounded cobbles and boulders and occasional glass and porcelain fragments; slightly moist. Some concrete rubble at approx. 6 ft. Caving below 6 ft; test pit terminated because of caving. No ground water encountered	

TP-304	GSE 1946'	July 13, 1994
0-9.0'	Very loose, gray-brown, slightly silty, gravelly SAND (FILL), with occasional rounded cobbles and boulders, organic debris, including wood fragments; slightly moist. Could probe 2-3 ft. at 6 ft. below ground surface. Extensive caving below approx. 2.5 ft.; test pit terminated because of caving. No ground water encountered	

TP-305	GSE 1947'	July 13, 1994
0-1.0'	Medium dense, dark brown, silty, gravelly SAND (FILL); dry.	
1.0-8.0'	Medium dense to dense, gray-brown, slightly silty, gravelly SAND (NATIVE), with occasional rounded cobbles; moist, weak cementation. Could probe 4-5 ins. at 5 ft. below ground surface. West side of trench caving due to 1 in. galvanized water line exposed at 5.5 ft. deep. No ground water encountered	

TP-306	GSE 1946'	July 13, 1994
0-8.0'	Loose to very loose, light brown, slightly silty, sandy GRAVEL (FILL), with occasional rounded cobbles; dry to slightly moist. Extensive caving up to surface; test pit terminated because of caving. Could probe greater than 3 ft. at 4 ft. below ground surface. No ground water encountered	

TP-307	GSE 1948'	July 13, 1994
0-5.0'	Medium dense, brown, silty, gravelly SAND (FILL), with occasional rounded cobbles and boulders and large quantities of trash, including tin cans and metal scraps, wire, glass, ceramic and some wood; moist.	
5.0-10.5'	Loose, gray-brown, slightly silty, gravelly SAND (FILL), with occasional cobbles and boulders; moist. Could probe approx. 4-6 ins. with 1/2 in. diameter steel rod at 3.5 ft below ground surface. Caving below 4 ft.; test pit terminated because of caving. No ground water encountered	

TP-308	GSE 1948'	July 13, 1994
0-9.0'	Loose, brown-gray, slightly silty, gravelly SAND (FILL), with occasional rounded cobbles and boulders and broken glass, metal and steel wire scraps; slightly moist. Could probe 1-1.5 ft. with 1/2 in. diameter steel rod at 6 ft. below ground surface. Caving below 2 ft.; test pit terminated because of caving. No ground water encountered	

TP-309	GSE 1947'	July 13, 1994
0-9.0'	Loose, gray-brown, slightly silty, gravelly SAND (FILL), with occasional rounded cobbles; moist. Could probe approx. 2 ft at 5 ft. below ground surface. Caving below 2.5 ft.; test pit terminated because of caving. No ground water encountered	