HPT-06/SC

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DESIGN BRIEF

CITY OF COQUITLAM

Pipeline Road Pump Station

ASSOCIATED ENGINEERING

November 1999

ASSOCIATED ENGINEERING

Al Geoghegan
City of Coquitlam
3000 Guildford Way
Coquitlam, B.C.
V3B 7N2

November 17, 1999
File: 962633

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Re: STRUCTURAL AND SEISMIC UPGRADE DESIGN PIPELINE ROAD PUMP STATION

Dear Mr. Geoghegan:

We are pleased to submit two copies of the design brief for the above noted project for your records.

We would be available to discuss the contents of the report with you at your convenience.

Respectfully submitted,

ASSOCIATED ENGINEERING (B.C.) LTD.

Dal B Harr

Dale B. Harrison, P.Eng.
Structural Engineer

Dal B. H
DALE HARRISON
Nov 23/99
ENGINEER

DBH/adv

Enclosure

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INTRODUCTION

SECTION 1

1.1 LOCATION AND DESCRIPTION

Pipeline Road Pump Station is located at the intersection of Robson Drive and Pipeline Road.

The Pipeline Road Pump Station is 14 m x 10 m x 4.8 m high reinforced concrete semi-buried structure which was originally constructed in 1988 and extended in 1992. It is an L-shaped in plan with a dominant dimension in the east/west direction. On the north side, the building penetrates into the slope approximately 3.5 m. The roof is partially backfilled on the north side with fill varying from 500 mm to 1500 mm deep. The remainder of the roof is exposed concrete.

In 1993 the pump station was expanded with a 3.0 m x 6.6 m extension to the east end and a 5.5 m x 3 m x 3.5 m high extension to the west end. Both structures were reinforced concrete to match the original construction. The original structure was also modified by removing a portion of the wall at the south end to permit the existing overhead crane to operate the full length of the pump station.

The roof of the building is 175 mm thick and is vertically supported on four sides by the walls. In addition it is supported by an upstand beam running from east to west, and by an underslab beam oriented north-south. The roof to wall connection is monolithic which provides lateral support for the earth-retaining walls. The walls are 200 - 250 mm thick and are typically reinforced on the inside face only. The foundations are 450 mm wide x 250 mm deep strip footings. The concrete floor is a 150 mm thick slab reinforced with wire fabric. The building contains electric pumps and ancillary equipment including and overhead crane and standby power.

1.2 DESIGN OBJECTIVES

The design objectives were as follows:

.1 To upgrade the structural capacity of the existing concrete roof to support the earth fill based on the information obtained from a pachometer survey.

.2 To mitigate the leaks in the existing roof slab.

.3 To design structural and seismic upgrading which would minimize impact on the adjacent property owners.

.4 To upgrade the Pipeline Road Pump Station to remain operational for the 1:475 year event with an importance factor of 1.0 and not collapse in the 1:1000 year event with an importance factor of 1.5. This provides an effective level of protection which is similar to that adopted by Greater Vancouver Water District for critical elements of their water system.

1.3 EVALUATION OF OBJECTIVES

The following objectives were achieved by the design:

.1 The structural capacity of the existing concrete roof was upgraded to support the roof earth fill.

.2 The roof of the station was waterproofed by the installation of a waterproof membrane and drain rock drainage layer.

.3 The station was upgraded for the 1:1000 year event with I=1.5, by the provision of soil anchors at the floor slab level to prevent global sliding and the rear wall was upgraded to resist static and dynamic earth pressures.

.4 The station non-structural items including the overhead crane, emergency generator, fuel tank, and some piping components were anchored to enable the station to remain functional in a 1:475 year event earthquake.

DESIGN METHODS AND ASSUMPTIONS

SECTION 2

2.1 GEOTECHNICAL

AGRA has prepared a geotechnical report comprising recommendations on parameters required for structural analysis. They have prepared peak "firm ground" accelerations for 1:1000 and 1:475 year events. The 1:1000 year event was used for stability and strength analysis, as outlined in the report.

Due to limited access, no investigation was undertaken of the fill behind the pump station wall and the geotechnical report is based on the assumption that the backfill is drained. The geotechnical report is bound in Appendix A.

2.2 STRUCTURAL ANALYSIS

The structural analysis has been based on the information contained in the record drawings supplied by the City of Coquitlam, the pachometer survey, and the geotechnical report. Load factors and loading combinations for dead loads, live loads, soil loads, and dynamic loads have been applied in accordance with the provisions of the National Building Code - 1995 and CAN/CSA A23.4-M94.

Structural analysis was conducted for both static and seismic load conditions. The building retains approximately 6 m of fill on the north side. It also supports 0.5 to 1.5 m of earth fill on a portion of the roof in addition to snow loads. The roof was analyzed as a two-way spanning slab supported by the walls and the upstand beam. The upstand beam transfers the gravity loads into the end walls and a north-south underslab beam at the junction of the original and extended portion of the pump station. The upstand beam is a continuous beam with positive reinforcement within the roof slab. It was analyzed as a rectangular beam for positive moments and as a T-beam for negative moments.

The buried walls of the building resist lateral earth pressures. The walls were analyzed as two-way spanning slabs for both active and at rest pressures. The active pressures require some lateral movement and the at rest pressures assume rigid walls. The station walls are semi-rigid and therefore the actual soil pressures will be somewhere between active and at rest.

When subjected to earthquake ground motions, earth pressures increase due to dynamic effects. The soil static and dynamic forces were calculated in accordance with the information provided by the geotechnical consultant assuming a semi-rigid wall with limited flexibility. Although there is no evidence of distress in the wall retaining the existing fill, it was marginal under static loads and potentially overloaded under dynamic earth pressures. We upgraded the pump station rear wall with vertical stiffeners designed to transfer the lateral forces into the roof and floor slab.

The global stability of the station was also evaluated based on the lateral pressures and friction coefficients provided by AGRA. Because the station has large unbalanced soil loads there is potential during an earthquake for the station to slide. Based on our analysis a movement of approximately 100 mm was predicted which would render the piping connections unserviceable. We upgraded the pump station by designing a reaction beam and soil anchors to resist sliding under the design earthquake.

2.3 SUMMARY OF STRUCTURAL ANALYSIS

Table 2.1 at the end of this section summarizes the results of the structural analysis prior to upgrading with deficiencies identified for specific critical elements. The roof elements are analyzed based on existing soil loads.

2.4 NON-STRUCTURAL ELEMENTS

We have also reviewed the adequacy of the non-structural components to withstand the design earthquake motions. These components, although not critical for the stability and strength, enable the pump station to continue to supply water after an earthquake. Mr. Sasa D. Popovic, E.I.T. conducted a site survey on April 04, 1997 and June 10, 1997, to gather data required for the evaluation of the non-structural components. We used a standard check list, provided by the client, as a basis for checking the adequacy of the non-structural components. (See Appendix B)

2.4.1 Hoisting Equipment

The Pump Station is equipped with a 910 kg capacity single-girder top-running crane. The crane has W200 x 36 steel runway-beams supported on steel brackets bolted to the concrete wall.

We upgraded the following items:

.1 We installed hooks, anchored to the west wall, to secure the winch chains when the crane is not in operation. The operational staff must park the crane adjacent to the west wall and place the chain in the hook for the system to be effective.

.2 We installed vertical stops on the end trucks to prevent the crane from dislodging from the crane rails during an earthquake.

2.4.2 Motor Control Panel

The existing MCC was anchored at the base to the floor and at the top to the wall. We evaluated the anchorage and it is adequate.

2.4.3 Emergency Standby Power

The station requires a reliable alternate power source to supply power to the pumps in the event of an earthquake. We evaluated the existing standby diesel generator and ancillary equipment to determine if these will meet this operational objective.

The station is equipped with a 320 kW standby generator which weighs 2746 kg and is supported by four LO-REZ (BR2-L5W(D)5-1600) seismic isolators. We supplemented the existing vibration isolators with seismic snubbers to ensure that the standby generator will remain operational in an earthquake.

The fuel lines had a flexible connection at the junction with the generator which allows them to accommodate differential movements induced by the design earthquake. The batteries were restrained to prevent toppling or sliding in an earthquake.

The pump station has a 2275 L above ground fuel tank. The City of Coquitlam personnel (see fax dated April 07, 1997, sent by Mrs. Lisa Knight) have confirmed that assuming a full tank, the generator would be able to supply power design engineer expect that it will perform adequately with very limited movement in the design earthquake. This is based on the performance of geogrid reinforced walls in California in past earthquakes. We also independently checked the overall stability of the wall and confirmed that it is stable under static and dynamic conditions.

The PRV discharge pipe does not have a flexible coupling at the interface of the pump station. However the discharge pipe is normally empty and therefore it is not essential to the post-disaster function of the pump station. There is some potential for the pipe to crack the wall locally if it moves differentially, however the flange adaptors inside the pump station have adequate flexibility to accommodate the expected rotation without damage to the surge pressure reducing valve. Therefore, a flexible connection was not installed at this location.

The main discharge pipe also extends through the south wall into the backfill retained by the lockblock wall at a lower level. At this level, we do not expect the differential movements between the pump station and retaining wall backfill to exceed 10 mm. The welded steel pipe should have adequate flexibility and strength to accommodate this movement. The pipe has a flange adaptor inside the wall which will allow it to rotate if the lateral movement causes the pipe to crack the south wall of the pump station. Due to the expense of removing the lock-block fill this pipe was not upgraded.

2.6.3 Yard Piping

Travelling seismic waves produce differential ground strains which result in axial and bending stresses in buried pipelines. We have evaluated the strains in the buried ductile iron piping by the Newmark method using the ground shear wave velocity provided by AGRA, for the Noons Creek Reservoir project. Based on our evaluation the underground ductile iron piping has adequately ductility and flexibility to accommodate the predicted movements. The cast iron fittings and valves outside the pump station are more vulnerable than the ductile iron piping and there is some question as to their adequacy. However, replacing them would be expensive and should be part of an overall plan to address piping vulnerability in the entire water supply system.

Table 2-1

Results of Seismic Analysis

Element or Failure Mode	Ultimate Capacity	Demand		CDR	Comments
		Elastic	Design		
1. Roof Slab					
1.1 Bending (kNm/m')	55.4		43.4	1.28	Gravity loads
1.2 Shear (kN/m')	90.4		62.9	1.44	
2. Roof Upstand Beam					Gravity loads
2.1 Bending (kNm)	159		366.5	0.43	Serviceability (crack control) state
2.2 Bending (kNm)	294.5		366.5	0.8	Ultimate state
2.3 Shear (kN)	191.5		186.9	1.02	
3. Roof North-South Beam					Gravity Loads
3.1 Bending (kNm)	493.4		525.2	0.94	
3.2 Shear (kN)	269.2		223.4	1.21	
4. North Wall					Drained backfill assumed
4.1 Bending vertical (kNm/m')	44.1	34.3	51.5	0.86	Static active case
4.2 Bending horizontal (kNm/m')	34.2	13.9	20.9	1.64	Static active case
4.3 Bending vertical (kNm/m')	44.1	62.2	93.3	0.47	Static at rest case
4.4 Bending horizontal	34.2	25.2	37.8	0.9	Static at rest case

Element or Failure Mode	Ultimate Capacity	Demand		CDR	Comments
		Elastic	Design		
4.5 Bending vertical (kNm/m')	44.1	186	111.6	0.4	1/1000 EQ, 1=1.5, U=0.6, R=1
4.6 Bending horizontal (kNm/m')	34.2	75.8	45.5	0.75	1/1000 EQ., 1=1.5, U=0.6, R=1
4.7 Shear (kN/m')	112.2	35.6	53.4	2.1	Static active case
4.8 Shear (kN/m')	112.2	64.4	96.6	1.16	Static at rest case
4.9 Shear (kN/m')	112.2	173.9	104.3	1.08	1/1000 EQ., 1=1.5, U=0.6, R=1
5. Stability					Limit state design
5.1 Static Sliding (kN)	1939.7	1503	2254.5	0.86	
5.2 Dynamic Sliding (kN)	1939.7	4512	4512	0.43	
5.3 Dynamic Overturning (kNm)	10333.5	8660	8660	1.19	

Note: C/D > 1 acceptable
C/D < 1 does not meet design criteria

SECTION 3

COST ESTIMATES

3.1 COST ESTIMATES

Table 3.1 summarizes cost estimates per item.

Table 3-1

Summary of Cost Estimates

Item	Cost Estimate
Remove and stockpile 120 cm fill, including removal of existing fencing, shoring, and hoarding.	$11,800.00
Reinforce existing upstand roof beam.	$10,000.00
Strengthen existing rear wall of pump station.	$13,000.00
New roof membrane with tapered insulation.	$9,800.00
Drains, reinstall crib wall and place fill.	$12,900.00
New security, cedar fence, and landscaping.	$3,700.00
Install ground anchors and reaction beam.	$35,000.00
Install sign and hoist chain hook.	$200.00
Install vertical restraint on bridge crane.	$1,000.00
Install seismic snubbers.	$8,000.00
Batteries restraint.	$500.00
Fuel tank restraint.	$2,000.00
Install restraint on PRV discharge.	$500.00
Securing the utility rack.	$300.00
Subtotal	$108,700.00
20% Contingency	$21,740.00
Engineering Supervision	$8,000.00
7% G.S.T.	$9,690.80
Total	**$148,130.80**

CONSTRUCTION DRAWINGS

SECTION 4

The following is the list of construction drawings used for this project.

- Title Page
- 2633-1-101 Site Plan
- 2633-1-102 Plans, Section & Details
- 2633-1-103 Sections & Details

APPENDIX A

AGRA EARTH AND ENVIRONMENTAL LIMITED GEOTECHNICAL SEISMIC REVIEW OF THE PIPELINE ROAD PUMP HOUSE STATION

AGRA Earth & Environmental
ENGINEERING GLOBAL SOLUTIONS

FACSIMILE TRANSMITTAL

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Phone No. (604) 294-3811
Fax No. (604) 294-4664

TO: Sasa D. Popvic
COMPANY: Associated Engineering
FAX NO.: 291-6183
SENDER: Wayne Quong HARD COPY TO FOLLOW: No
FAX OPERATOR: FILE NO. VG 07148
NO. OF PAGES: 2 (including this page) DATE: 25 April 1997

SUBJECT: Re: Geotechnical Seismic Review
Pipeline Road Pumphouse Station

The following is a preliminary summary of the geotechnical seismic input parameters for your review of the seismic response of the Pipeline Road pumphouse station. A report will follow early next week.

1. Peak "firm ground" acceleration levels.

TABLE 1
SUMMARY OF SEISMIC HAZARD ANALYSIS

Design Earthquake	475 yr	1000 yr
Peak Horizontal Grd. Accel. (g)	0.21	0.27
Peak Horizontal Grd. Velocity (m/sec)	0.21	0.28

2. Static and dynamic earth pressures to be considered in pumphouse wall design.

Assuming relative rigidity of the foundation walls caused by the restraining influence of the concrete roof and end walls. Static pressures can be assumed to be "at rest" pressures, with $K_o = 0.5$

Dynamic pressures can be estimated using "rigid wall" earth pressure theory where "zero" wall displacements are assumed. The maximum dynamic lateral force (acting in addition to static earth pressure forces) can be conservatively estimated to be,

$$F_d = \gamma H_T^2 A_{max}$$

where γ is the unit weight of the wall backfill (a value of 20 kN/m^3 can be assumed), H_T is the total

DRAFT

height of the backfill at the wall location, A_{max} is the peak ground surface acceleration, expressed as a fraction of gravity. A_{max} values for the 475 year and 1000 year events can be assumed to be equal to the firm ground PGA values presented above. An approximate parabolic pressure distribution can be assumed. To compute the dynamic force exerted on the wall, the portion of the pressure distribution above the wall can be neglected.

Where deflections at the top of the wall exceed 0.5% of the wall height (approx. 25 mm) dynamic pressure can be estimated using the Mononobe-Okabe earth pressure theory. The force exerted on the wall by the soil mass is given by the expression.

$$P_{AE} = 1/2\gamma H^2 \cdot K_{AE} + \gamma d K_{AE}$$

where, H= height of wall (4.78 m) and d= depth of wall below top of backfill (1.6 m)

It is sufficient to assume that the active seismic pressure acts a uniform distribution, with the resultant of the seismic active earth pressure located at the mid height of the wall. Where the wall can be designed to accommodate some additional movement, the K_{AE} value can be reduced based on procedures proposed by Richards and Elms (1979). Assuming drained backfill conditions, the following table presents K_{AE} values vs displacement:

Displacment (mm)	Seismic Pressure Active Coefficient K_{AE}	
	475 yr Earthquake	1000 yr Earthquake
25	0.348	0.383
50	0.335	0.363
75	0.329	0.353
100	0.325	0.347

In a similar manner, the A_{max} value used to estimate seismic rigid wall pressures can be reduced based on the above procedures. The following table presents A_{max} values vs displacement:

Displacment (mm)	Seismic Pressure Rigid Wall Coefficient A_{max}	
	475 yr Earthquake	1000 yr Earthquake
0	0.210	0.270
3	0.186	0.260
6	0.157	0.219

3. Static and dynamic vertical bearing pressures for the pumphouse footings.

Maximum static allowable bearing pressure: 150 kPa
Maximum dynamic allowable bearing pressure: 225 kPa

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4. Foundation sliding resistance

Coefficient of dynamic passive earth pressure K_{PE} (small deflection): 2.0
Coefficient of friction along the base of the foundation: 0.45

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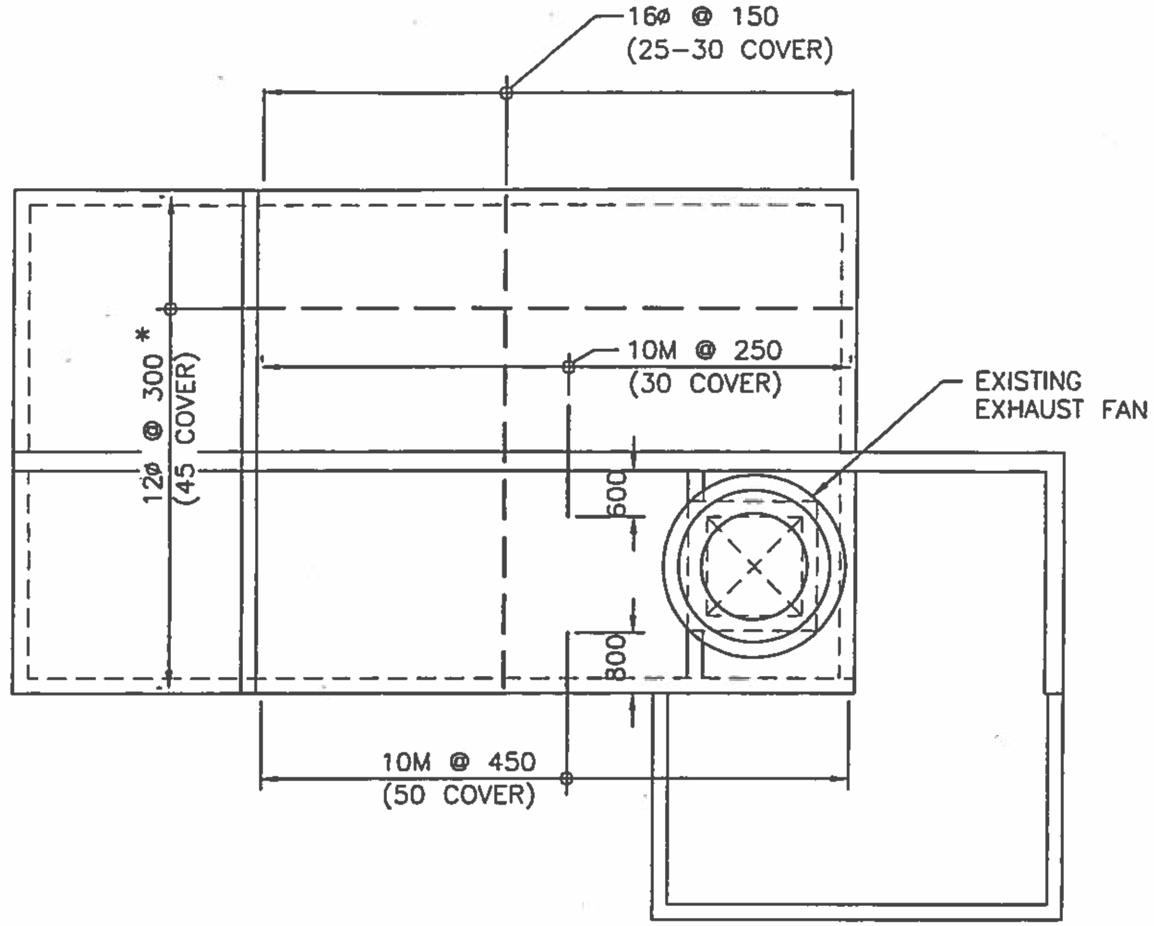
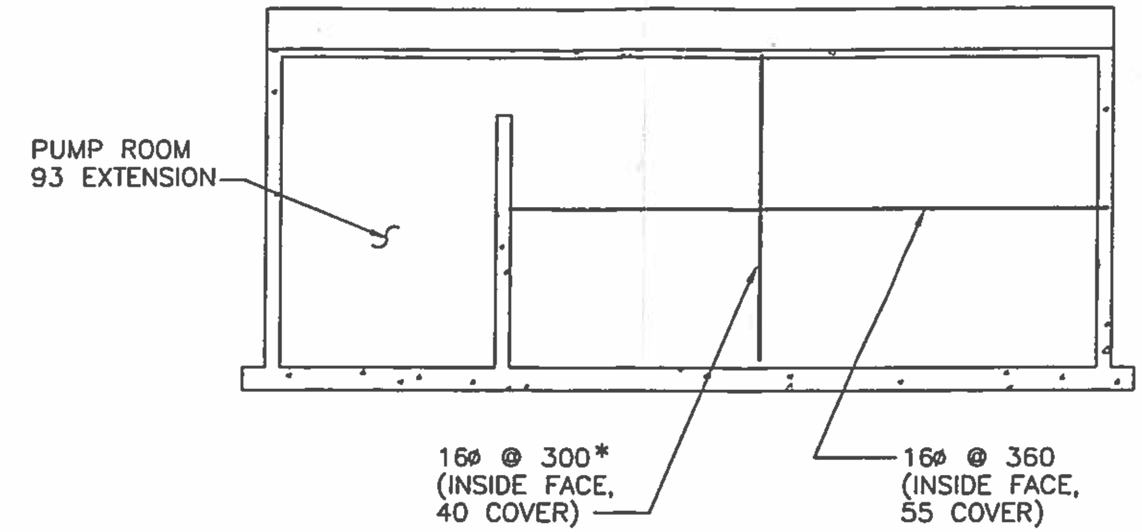
PACHOMETER SURVEY RESULTS

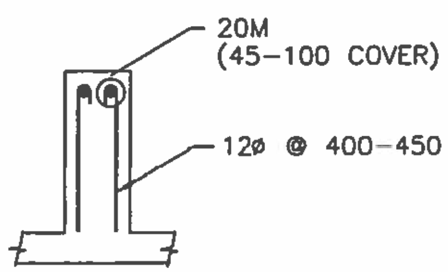
APPENDIX B



ROOF SLAB



NORTH WALL



ROOF UPSTAND BEAM

NOTE:
* ACTUAL SPACING
VARIES FROM 250–350

CONCEPTUAL
ONLY

LEGEND:

——— TOP REINFORCEMENT

– – – BOTTOM REINFORCEMENT

CLIENT:
CITY OF COQUITLAM

PIPELINE ROAD PUMP STATION

ASSOCIATED ENGINEERING

STRUCTURAL AND SEISMIC UPGRADE
PACHOMETER SURVEY RESULTS

DESIGN: S.D.P.		DATE: MAY. 1997	
CHECK:		SCALE: N.T.S.	
DRAWN: M.L.		DWG. No.	
JOB No. 962633		2633–SK–204	0

ASSOCIATED ENGINEERING (B.C.) LTD. LETTER DATED JANUARY 10, 1997

APPENDIX C

January 10, 1997
File: 962633

Frank Quinn, P.Eng.
Assistant City Engineer, Design and Construction
City of Coquitlam
1111 Brunette Avenue
Coquitlam, B.C.
V3K 1E9

Re: PIPELINE ROAD PUMPSTATION

Dear Mr. Quinn:

1. TERMS OF REFERENCE

Associated Engineering was retained by the City of Coquitlam to investigate cracking and leakage of the existing Pipeline Road Pump Station. The original proposed scope of the work was as follows:

.1 Assess the structural capacity of the existing concrete roof to support the earth fill (this included a pachometer survey)
.2 Determine the cause of roof leakage
.3 Recommend leakage retrofit and if necessary load reduction on the roof.
.4 Submit letter report with recommendations and cost estimates for leakage retrofit and structural options as required.

Subsequent to our initial proposal, Coquitlam staff decided that the roof fill would be removed, therefore the terms of reference were revised to delete the pachometer survey and structural assessment of the existing roof.

2. AVAILABLE DATA

The City of Coquitlam provided Associated Engineering with the construction drawings "Expansion of the Pipleline Road Pump Station", Sheets 1 to 17 inclusive, dated January 15, 1993 prepared by Kerr Wood Leidal Associated Ltd. No drawings were available for the original pump station.

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3. DESCRIPTION OF STRUCTURE

The original pump station, constructed in 1988, consisted of a reinforced concrete structure approximately 6.6 m x 8.1 m x 4.5 m high, and a 3.4 m x 3.0 m extension at the north end. The station was designed for approximately 600 mm of earth fill on the west half of the roof. The remainder of the roof was exposed concrete. An additional 900 mm of earth fill was placed on the approximately 1 metre strip of roof on the east (uphill) side. Therefore the west wall retains approximately 6 metres of fill.

In 1993, the pump station was expanded with a 3.0 m x 6.6 m extension to the south end and a 5.5 m x 3 m x 3.5 m high extension to the north end. Both structures were reinforced concrete to match the original construction. The original structure was also modified by removing a portion of wall at the south end to permit the existing overhead crane to operate the full length of the pump station.

The station roof has recently been shored with steel pole shores and scaffold frames due to concern regarding the roof flexural cracks and visible roof deflection.

4. CONDITION SURVEY

Mr. Dale B. Harrison, P. Eng, and Mr. Oliver Loucks of Associated Engineering inspected the site and did a visual survey of the interior cracks on December 6, 1996. There are several cracks in the roof and wall of the pump station which are leaking or have leaked in the past. The primary cracks are:

.1 Roof crack at the junction of the south addition roof beam and the roof beam in the original building;

.2 Two roof cracks in the north addition; radiating from the north east corner of the original pump station;

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.3 Roof crack at about mid length of the original building extending from the east wall to approximately midspan;
.4 Wall leak at the junction of the north addition and the north east corner of the original building;
.5 Non-leaking flexural cracks, in the underside of the roof slab below the earth fill.

The visible cracks were measured and most did not exceed 0.1 mm wide with the maximum width being 0.2 mm. There is approximately 30 mm of deflection in the roof of the original pump station.

5. DISCUSSION

The addition to the pump station has adequate structural capacity to support the existing roof and wall loads. The capacity of the existing pump station roof was not checked as the structural drawings were not available and it was decided that a pachometer survey to confirm reinforcing patterns was unwarranted in view of the decision to remove the roof fill. An overall stability check indicates that the station is marginally stable with existing static soil loads. A detailed seismic check was outside the scope of this assignment, however, based on our cursory review it is anticipated that the design earthquake could overload the existing structure or cause it to slide. Although no evidence of distress in the wall retaining the fill was observed, we recommend removing enough earth fill to reduce the overall earth pressure on the station west wall to be no more than that from the original 600 mm design fill. This will also ensure that the overall stability of the structure is not significantly reduced due to the removal of the roof dead load. Therefore it is recommended that the top of the retaining wall be located a minimum of 1.5 metres clear of the existing station east wall.

We believe the majority of cracks in the roof of the station which have leaked are caused by shrinkage and temperature stresses. They are generally in the direction perpendicular to the main reinforcing steel and are located in the areas of the roof which are exposed to the greatest thermal ranges. The one leaking crack which may be partially caused by load is the joint at the interface of the

.../4

new and existing beam. This crack may be partly due to the inverted T-beam spanning the length of the station causing the roof to open in tension.

There are some non-leaking hairline flexural cracks visible in the underside of the roof slab under the filled area. All the cracks at the time of our inspection were less than 0.2 mm in width which means there is no risk of corrosion to the reinforcing as they meet the CSA A23.3 crack limit of 0.4 mm for interior exposure and 0.33 mm for exterior exposure.

There is approximately 30 mm of sag in the centre of the roof above the steel monorail beam. This deflection (assuming the slab soffit was cast level) does not exceed the limit of total long term deflection permitted by the CSA A23.3 (L/240=34 mm).

Therefore although there is the possibility of an overload on the roof under the current earth loading it is not conclusive from our preliminary analysis or the evidence we gathered from our site investigation. In our opinion the majority of the cracks were caused by differential shrinkage and temperature movements and are not load related.

6. RECOMMENDATIONS

The alternatives for fixing the roof leaks are epoxy injection and a waterproof membrane. We would recommend the installation of a water proof membrane as it protects the structure from moisture and eliminates the risk of further leakage due to cracks opening under future thermal stresses. Providing the earth fill is permanently removed from the roof, a conventional roofing system such as a 2 ply modified bitumastic membrane will provide a reasonably economical roof with a 20 year life-expectancy.

Based on instructions from the City of Coquitlam we have prepared the enclosed drawings (# 2633 - SK - 101 to 2633 - SK - 103 inclusive) indicating the fill

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permanently removed from the pump station with the timber crib wall, security fence, and decorative cedar fence relocated to suit.

7. COST ESTIMATE

.1	Remove and dispose of 300 cm fill, including shoring and hoarding	$13,200
.2	New crib wall, drains, catch basins, place fill	$25,500
.3	150 cm new free draining granular fill	$3,800
.4	New security and cedar fence and landscaping	$4,900
.5	New roof membrane with tapered insulation	$9,800
	Subtotal	$57,200
	10% Contingency	$5,700
	Engineering Supervision	$5,000
	7% GST	$4,800
	Total	$72,700

Please do not hesitate to contact the undersigned if you have any queries regarding the project. Thank you for the opportunity of working on this project.

Yours truly,

Dale B. Harrison, P. Eng
Structural Engineer

Enclosure

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