

G.I.-30 SEPT. 1976

GEOCRES No. SOM13-89

DIST. 6 REGION

W.P. No. _____

CONT. No. _____

W. O. No. 29-11001

STR. SITE No. 37-1298

HWY. No. 400

LOCATION Langstaff Rd. Underpass

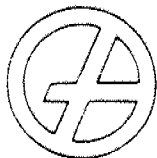
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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____

FILE COPY

GEORES No
30M13-89



Golder Associates Ltd.

CONSULTING ENGINEERS

REPORT TO
MCCORMICK RANKIN & ASSOCIATES LIMITED

**DYNAMIC MONITORING OF PILES
LANGSTAFF ROAD BRIDGE
OVER HIGHWAY 400
TOWN OF VAUGHAN, ONTARIO**

Distribution:

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April 1991

901-1404



Golder Associates Ltd.

CONSULTING ENGINEERS

April 25, 1991

Our ref: 901-1404

McCormick Rankin & Associates Limited
2655 North Sheridan Way
MISSISSAUGA, Ontario
L5K 2P8

ATTENTION: Mr. J.L. Malcolm, P. Eng.

**RE: DYNAMIC MONITORING OF PILES
LANGSTAFF ROAD BRIDGE OVER HIGHWAY 400
TOWN OF VAUGHAN, ONTARIO**

Dear Sirs:

This letter presents the results of dynamic monitoring and testing of piles carried out by Anna Geodynamics Inc. at the above site between October 4 and November 12, 1990 during which time eighteen piles were tested. The purpose of the testing was to establish appropriate driving criteria for the design loads of the HP 310x110 piles driven for the support of the proposed bridge structure.

The site is located on Highway 400 between Highway 7 and Rutherford Road in the Town of Vaughan, Ontario (see Figure 1). The project involves the construction of a three span bridge with total length of about 85 m. Based on the results of our investigation at the bridge site (our report number 881-1437 dated February, 1989), the subsoils at the site, in general, consist of a complex succession of silty clay, sandy to clayey silt till, sand and silt deposits of variable consistency overlying hard clayey silt to silty clay till. The depth, composition and consistency of the underlying cohesive deposits is variable across the site and the behaviour and/or achievable set for piles driven into these deposits was anticipated to be different at each of the pier and abutment locations.

TEST RESULTS

A total of eighteen steel H-piles were tested by Anna GeoDynamics Inc. under our supervision. The testing was carried out on four piles installed at the east abutment, on seven piles installed at the west pier and on seven piles installed at the west abutment.

During the testing, the dynamic strain and acceleration in the pile during driving are measured by strain gauges and accelerometers mounted on the surface of the pile. Through the pile driving analyser, the time history for dynamic strain and acceleration of the pile head are recorded for each hammer blow. The field records are processed by means of the DATPRO program which determines the energy transferred to the pile from the hammer, the impact force in the pile and estimates the static pile capacity (Case Method Estimate, CMES). The field records are stored for further analysis by the CAPWAP program (Case Pile Wave Analysis Program).

CAPWAP analyses were performed on seven piles to calibrate the bearing capacity determined by the Case Method Estimate (CMES). The test results are summarized in Table 1 enclosed at the end of the text. The foundation and piling layout is shown on Figure 2. The plots of the pile driving records based on the field records provided by McCormick Rankin are attached in Appendix A. The detailed dynamic monitoring test results are presented in Anna GeoDynamics Inc. report number 1, file number 141-J dated December 18, 1990, submitted under separate cover.

a) East Abutment

Pile installation was commenced on October 1, 1990, by Bermingham Construction Ltd. using a hydraulic hammer IHC Hydrohammer S-70 with nominal hammer energy set at 50 kJ. The dynamic monitoring was carried out on Piles D5, D10, D11 and D13 on October 4, 1990. Details of the monitoring results are given on the attached Table I. In summary, Piles D5, D10 and D11 were driven to similar depths but with variable final sets (17 to 40 blows per 0.3 m) and were terminated between Elevations 181.5 m and 182.7 m, within the sand deposit.

Testing of Piles D5 and D10 was carried out two days after installation while testing of D11 was completed one day after installation. On the day of the testing, Pile D13 was driven about 5 m deeper than the other test piles and was terminated at a higher initial set (80 blows per 0.3 m) within the silty clay deposit. This pile was tested initially immediately after driving and then again about 4 hours following completion of driving.

The nominal hammer energy was set at 40 kJ for the tests carried out on Piles D5, D10 and D11 while it was increased to 50 kJ (nominal) for the test on Pile D13. The dynamic measurements establish the energy actually transferred to the pile, the impact force delivered by the hammer and the maximum force in the pile. The dynamic monitoring test results indicate that about 85% to 95% of the energy was being transmitted to the piles during restrike.

The energy and force measurements are used to provide an estimation of the ultimate static pile capacity (the Case Method Estimate) of combined shaft and toe resistance. The Case Method Estimates indicate that the ultimate pile capacities for Piles D5, D10

and D11 which are terminated in the sand deposit vary from 1950 kN to 2170 kN. Although the sets at the end of driving of Piles D10 and D11 are substantially different, the load capacities of these piles are similar since the redrive sets during monitoring are about the same. The capacity of Pile D13 terminated in the silty clay deposit is indicated to be 1715 kN immediately following driving and 2010 kN after a four hour set up time.

The Case Method Estimate capacities are generally considered to be accurate to within 10 per cent of the actual load capacity with the variation being dependant mainly on the site subsoil conditions, the damping factors and the ease of driving. A further, and more detailed, calibration of the site parameters is accomplished by a CAPWAP analysis (wave mechanics computations) which is carried out on the input from one blow.

The results of the CAPWAP analyses carried out on Pile D10 indicate an ultimate capacity of 1750 kN, comprised of 1700 kN in shaft resistance and 50 kN in toe resistance. A value of 1850 kN has been given as the upper bound of the analysis. The CAPWAP analysis carried out on Pile D13 indicates an ultimate capacity of 2200 kN, comprised of 790 kN in shaft resistance and 1410 kN in toe resistance. The difference in shaft resistance values between the two piles is a function of the time available for soil set up around the pile after completion of driving. The capacity would likely increase with time by at least 900 kN based on the shaft resistance component of Pile D10.

b) West Pier

The dynamic monitoring was carried out on Piles A2, A4, A6, A7, A9, A12 and A13 on November 2, 1990. The nominal hammer energy was set at 50 kJ. Except for Pile A13, all of the piles tested were driven to similar depths (between Elevations 179.5 m and 180.9 m) and were terminated in the clayey silt till deposits. The piles were tested at one to four days after the end of initial driving.

The measured energy transfer from the S-70 hydraulic hammer was more variable during this testing than would normally be expected with this type of hammer. For all of the piles tested, the energy ratio was about 50 per cent to 65 per cent for the first blow during monitoring. During subsequent blows, the energy ratio was generally 75 per cent to 95 per cent which is more typical of this hammer. The results of these testings are somewhat distorted since the capacity is generally based on the first blow during monitoring.

The estimated ultimate pile capacities for Piles A2, A4, A6, A7, A9 and A12 vary from 1800 kN to 2600 kN. The capacity of Pile A13 is about 1400 kN. Since Pile A13 was tested at one day after end of initial driving, the capacity would likely increase with time.

The results of CAPWAP analyses carried out on Pile A6 indicate an ultimate capacity of 2320 kN, comprised of 2170 kN in shaft resistance and 150 kN in toe resistance. The CAPWAP analyses carried out on Pile A13 at the "beginning-of-restrike" indicates an ultimate capacity of 1065 kN in which 1025 kN and 40 kN are distributed to the shaft and toe resistance respectively.

c) West Abutment

The dynamic monitoring was carried out on Piles A1, A5, C2, C4, C5, C8 and C9 on November 2 and 12, 1990. The nominal hammer energy was set at 50 kJ. All of the seven piles tested were driven to depths ranging from 21 m to 31 m (tip elevations between 178.2 m and 186.9 m). The piles were tested at one to four days after the end of initial driving and Piles A1 and C2 were retested at eleven days after driving.

The measured energy ratio was generally between 75% and 95% which is typical of this hammer. The estimated pile capacities for all of the piles tested vary from 1400 kN to 2700 kN. Piles A1 and C2 were retested at eleven days after driving and the pile capacities increased substantially to about 1950 kN and 2115 kN respectively, indicating the increase in the pile capacity with time.

CAPWAP analyses were performed on Piles A1, C2 and C4. The analysis of Pile A1 indicates an ultimate capacity of 1950 kN, comprised of 1870 kN and 80 kN in shaft friction and toe resistance respectively. A capacity of 2270 kN in shaft resistance and 105 kN in toe resistance resulting in a total of 2375 kN was determined for Pile C2. The CAPWAP analyses carried out on Pile C4 indicates an ultimate pile capacity of 2780 kN comprised of 2660 kN in shaft resistance and 120 kN in toe resistance.

DISCUSSION

The driving criteria appropriate for the maximum loading of the piles was determined for each of the east abutment, west abutment and west pier based on our interpretation of the results of the dynamic monitoring. The maximum loadings provided are as follows:

East Abutment

Row A :	732 kN	SLS;	951 kN	ULS
Row B :	645 kN	SLS;	810 kN	ULS
Row C :	559 kN	SLS;	685 kN	ULS
Row D :	451 kN	SLS;	557 kN	ULS

West Pier

Rows A and B :	910 kN	SLS;	1070 kN	ULS
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West Abutment

Rows A, B and C :	666 kN to 752 kN	SLS;	875 kN to 955 kN	ULS
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The following driving criteria were provided for acceptance of the piles at the east and west abutments in order to achieve the above capacities:

- It should be ensured that the hammer energy is set at between 50 and 55 kJ during driving.
- The piles should be driven to at least Elevation 183 m.
- A minimum resistance of 17 blows per 0.3 m penetration must be achieved over the final 0.3 m of driving.
- The set should be monitored per 25 mm for a further 100 mm of penetration. If there is a decrease in the number of blows per 25 mm within the initial group of four, the pile should be driven further until there are four consecutive sets/25 mm which are equal to or greater than the previous.
- At least 20 per cent of the piles, and any pile with final set during driving of less than 20 blows per 0.3 m, should be retapped. If a retap set of less than 13 blows per 100 mm is obtained, the pile must be driven deeper.

For the west pier, a minimum resistance of 30 blows per 0.2 m of penetration was specified over the final 1 m of driving with retaps on at least 10 per cent of the piles. The retap set specified for acceptance was 5 blows for a maximum of 25 mm of penetration.

The maximum capacity measured for the piles driven at the west pier would not have been adequate for the maximum loadings of 833 kN SLS and 1323 kN ULS given for the east pier piles. Since it was considered that the driving conditions were not likely to improve sufficiently to achieve the required capacity, four additional piles were installed at the east pier to reduce the maximum loading to that of the west pier. The driving

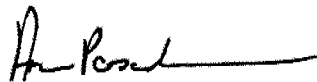
criteria established from the dynamic monitoring results for the west pier piles was then adopted for the east pier.

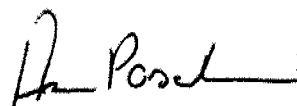
Based on a review of the driving records for the piles driven at the site subsequent to the testing, the sets measured during initial driving and the retap sets have satisfied the above specified driving criteria.

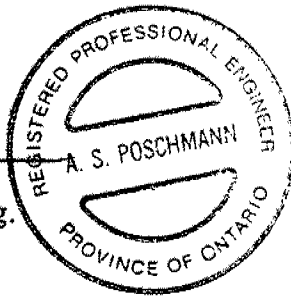
We trust that this report is sufficient for your immediate requirements. If you have any questions regarding the contents of this report, please do not hesitate to contact the undersigned.

Yours truly,

GOLDER ASSOCIATES LTD.


for K.S. Ho


A.S. Poschmann, P. Eng.



KSH/ASP/dh

Att: Table 1
Figures 1 & 2
Appendix A - Pile Driving Record Plots

TABLE 1
SUMMARY OF DYNAMIC TEST RESULTS

PILE	TIP ELEVATION (m)	EMBEDDED LENGTH (m)	DRIVING RESISTANCE (Blows/25 mm)		ULTIMATE PILE CAPACITY		ENERGY RATIO (%)
			FINAL SET	REDRIVE SET	CASE METHOD (kN)	CAPWAP (kN)	
East Abutment:							
D5	181.5	23.1	3.3	9	2165		75
				9	2170		85
				7	2150		95
D10	182.7	21.9	1.4	3	1955	1750	85
				3	2025		95
				3	2170		95
D11	182.6	21.9	2.7	4	1990		75
				4	2015		95
				3	1950		95
D13	176.9	28.2	6.7	13	1715		90
				13	1790		95
				10	2010	2200	90
West Pier:							
A2	180.9	25.0	6.3	9	1635		55
				9	1885		75
				6	1755		85
A4	180.8	25.1	5.6	10	2110		50
				10	2400		85
				10	2250		85
A6	180.8	25.1	4.0	5	2235	2170	65
				5	2395		85
				5	2405	2320	90
A7	179.9	26.0	3.5	6	2610		65
				6	2315		80
				6	2165		80
A9	179.5	26.4	3.1	4	1810		60
				4	1860		85
				4	1710		95
A12	179.4	26.5	5.8	3	1630		60
				3	1855		90
				3	1810		90
A13	171.7	33.0	2.8	3	1080	1065	50
				3	1395		95

TABLE 1 (cont'd)

SUMMARY OF DYNAMIC TEST RESULTS

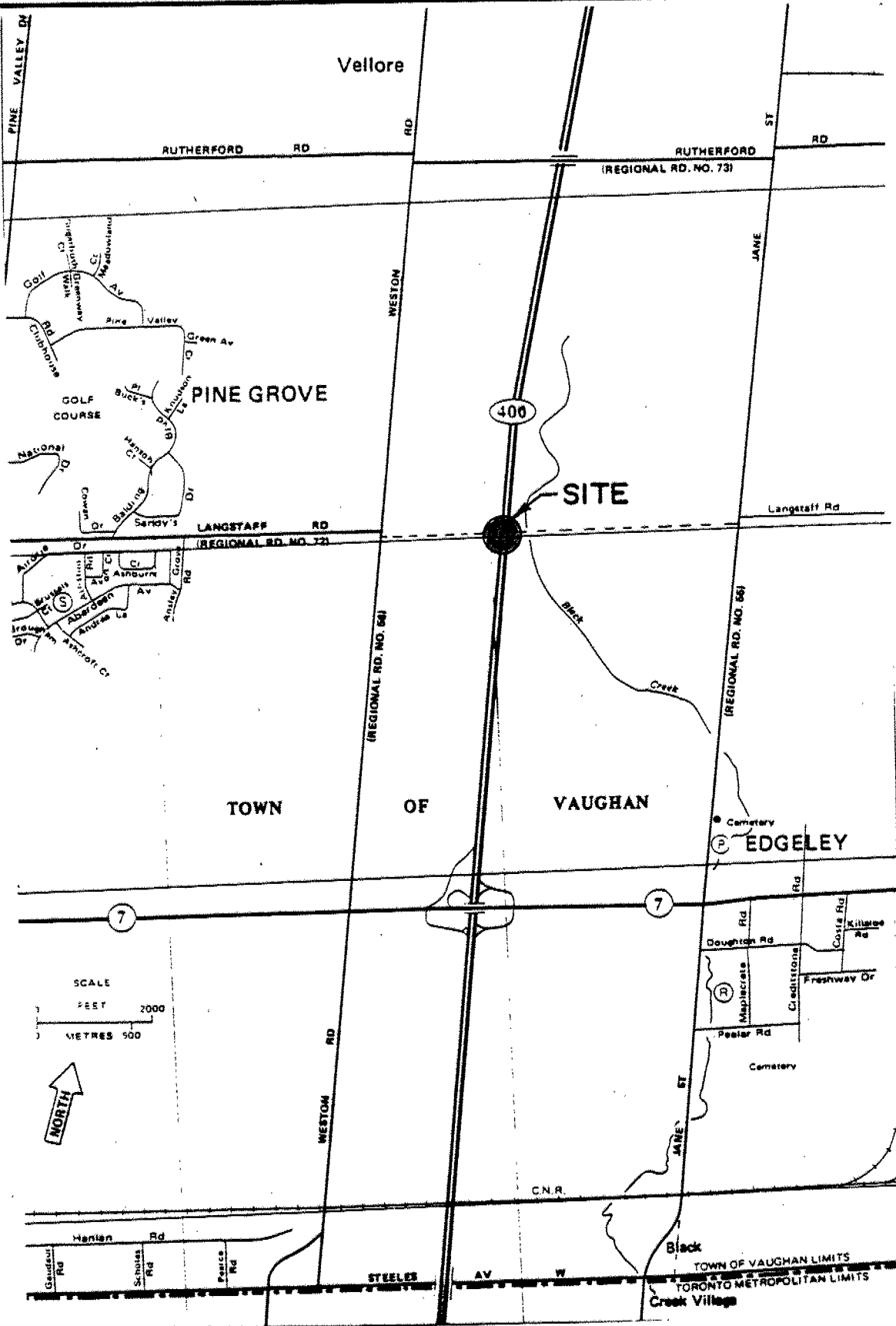
PILE	TIP ELEVATION (m)	EMBEDDED LENGTH (m)	DRIVING RESISTANCE (Blows/25 mm)		ULTIMATE PILE CAPACITY		ENERGY RATIO (%)
			FINAL SET	REDRIVE SET	CASE METHOD (kN)	CAPWAP (kN)	
West Abutment:							
A1	177.9	29.5	4.4	5	1110		50
				5	1550		80
				5	1510		85
				6	1960	1950	90
				6	2165		85
				6	1950		90
A5	177.9	29.6	2.5	4	1740		90
				4	1540		85
				4	1545		90
C2	179.7	29.5	8.5	7	1355	1360	60
				7	1575		90
				7	1550		95
				12	2250	2375	95
				12	2400		90
				12	2115		90
				12	2115		90
C4	186.9	21.6	3.9	13	2810	2780	80
				13	2690		85
				13	2445		75
C5	182.2	26.6	2.8	4	2180		75
				4	2055		85
				4	2105		95
C8	178.2	30.8	5.3	11	1775		75
				11	1750		80
				11	1750		90
C9	178.6	30.4	2.5	5	1420		80
				5	1400		85
				5	1260		80

Notes:

- 1) All piles tested with nominal hammer energy set at 50 kJ except for Piles D5, D10 and D11 of East Abutment (40 kJ nominal).
- 2) Final set refers to the driving resistance at the end of initial driving of the piles.
- 3) Redrive set refers to the driving resistance during retapping at the time of the dynamic monitoring.
- 4) All of the piles were driven by the IHC 570 hydraulic hammer.
- 5) Piles A9 and A12 (West Pier) and Piles A5 and C9 (West Abutment) were driven deeper after testing.
- 6) Pile A13 (West Pier) was tested immediately after initial driving and was retapped after full set-up time was complete.

SITE LOCATION PLAN

FIGURE 1



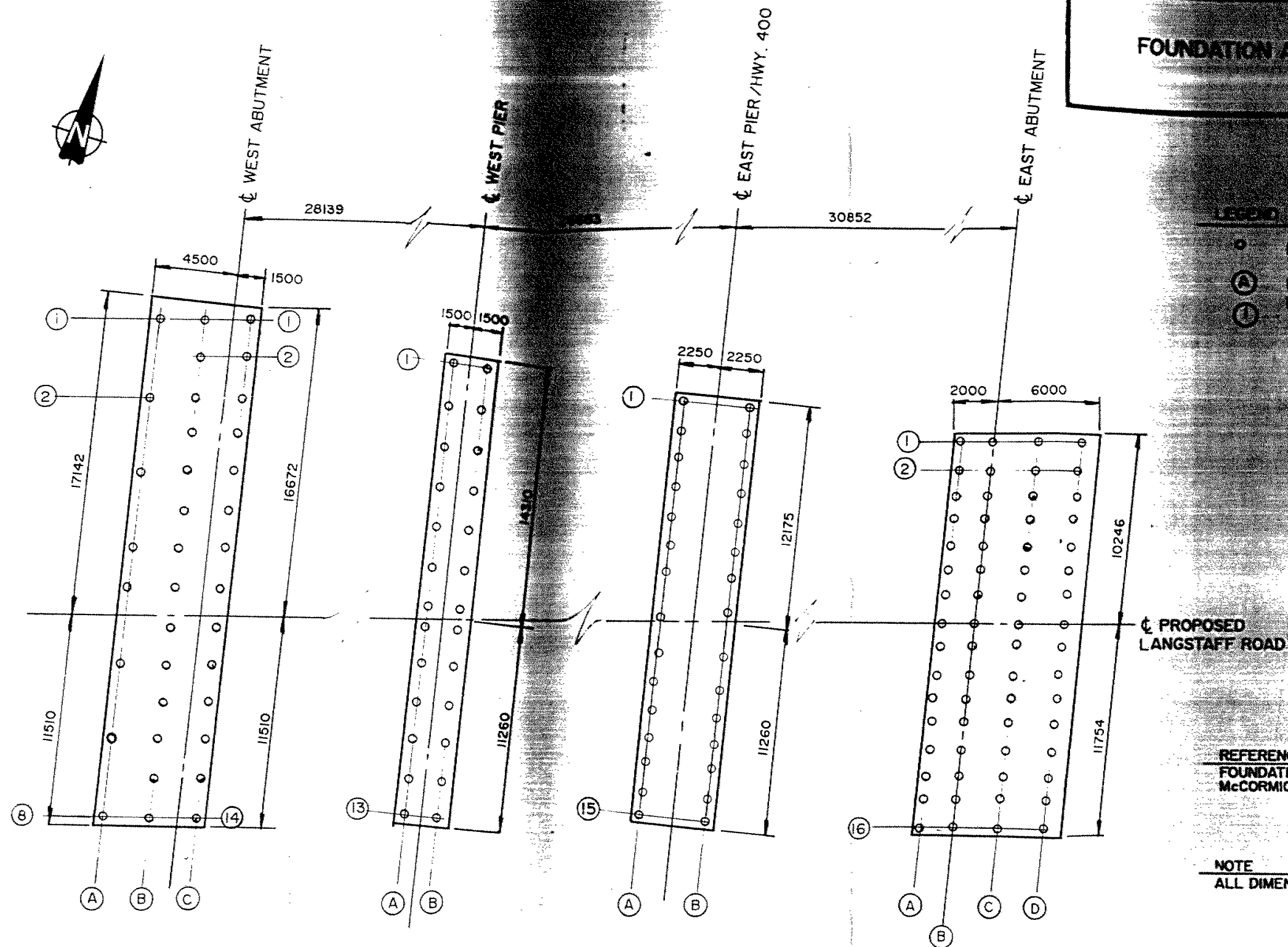
Date FEB. 1, 1991
Project 901-1404

Golder Associates

Drawn D.M.
Chkd. *byr*

FOUNDATION AND PILE LAYOUT

FIGURE 2



LEGEND

- HP 310 x 110 STEEL PILES
- ① ROW AND COLUMN IDENTIFICATION FOR PILE NUMBERING

REFERENCE

FOUNDATION AND PILING LAYOUT PLAN PREPARED BY
McCORMICK RANKIN

NOTE

ALL DIMENSIONS ARE IN MILLIMETRES

SCALE 1 : 250

Date FEB. 1, 1991
Project 901 - 1404

Golder Associates

Drawn D.M.
Chkd R.H.

APPENDIX A

PILE DRIVING RECORD PLOTS

April 1991

901-1404

TABLE A-1 (cont'd)

HWY 400/LANGSTAFF ROAD
SUMMARY OF PILE TIP CONDITIONS
(Obtained From Pile Driving Records)

EAST ABUTMENT									
PILE I.D.	DRIVEN LENGTH (m)	TIP ELEV (m)	FINAL SET (Blows/200 mm)	RETAP (Blows/25 mm)	PILE I.D.	DRIVEN LENGTH (m)	TIP ELEV (m)	FINAL SET (Blows/200 mm)	RETAP (Blows/25 mm)
A1	22.0	182.9	18		C1	21.8	183.1	28	
A2	22.6	182.4	16		C2	21.8	183.1	25	
A3	23.8	181.2	14		C3	21.8	183.1	29	
A4	26.2	178.9	25		C4	21.6	183.3	39	
A5	26.2	178.9	28		C5	21.6	183.3	29	
A6	25.9	179.2	25		C6	22.2	182.7	18	
A7	26.8	178.4	16		C7	22.0	182.9	21	5.1
A8	26.0	179.1	19	3.3	C8	21.6	183.3	30	
A9	25.8	179.3	17		C9	21.8	183.1	22	
A10	26.0	179.1*	18		C10				
A11	27.4	177.8	19		C11	21.8	183.1	21	
A12	28.0	177.2	19		C12	21.8	183.1	28	
A13	27.1	178.1	24		C13	22.0	182.9	16	
A14	27.2	178.0	16	2.9	C14	21.6	183.3	16	3.1
A15	22.0	182.9	16		C15	22.2	182.7	19	4.8
A16	22.0	182.9	22		C16	21.8	183.1	20	
B1	21.8	183.1	20	3.5	D1	21.6	183.3	27	3.5
B2	21.8	183.1	28		D2	21.9	183.0	17	
B3	26.2	178.9	23	3.1	D3	22.6	182.4	38	
B4	25.8	179.3	15		D4	22.3	182.7	18	
B5	26.0	179.1	14		D5	23.5	181.5	40	
B6	22.2	182.1	18		D6	22.3	182.7	30	
B7	21.8	183.1	25	3.8	D7	22.3	182.7	27	
B8	22.0	182.9	23		D8	22.6	182.4	24	5
B9	21.2		33		D9	22.9	182.1	23	
B10	21.8	183.1	29	5	D10	22.3	182.7	17	
B11	22.8	182.2	26		D11	22.3	182.7	32	
B12	21.8	183.1	17		D12	21.9	183.0	23	
B13	21.6	183.3	21		D13	28.3	176.9	80	
B14	21.8	183.1	21		D14		182.8	NOT RECORDED	
B15	21.6	183.3	29		D15	22.3	182.7	37	
B16	22.0	182.9	29		D16	22.6	182.4	48	4.3

TABLE A-1 (cont'd)

**HWY 400/LANGSTAFF ROAD
SUMMARY OF PILE TIP CONDITIONS**

(Obtained From Pile Driving Records)

EAST PIER				
PILE I.D.	DRIVEN LENGTH (m)	TIP ELEV (m)	FINAL SET (Blows/200 mm)	RETAP (Blows/25 mm)
A1	22.4	183.1	41	
A2	22.0	183.5	44	
A3	23.0	182.6	60	10.4
A4	23.0	182.6	49	6.3
A5	23.0	182.6	42	
A6	24.8	180.9	52	
A7	24.2	181.4	72	
A8	24.8	180.9	58	
A9	24.8	180.9	50	
A10	20.0	185.4	42	
A11	20.0	185.4	38	6.9
A12	20.0	185.4	40	
A13	20.0	185.4	43	
A14	20.2	185.2	40	8.3
A15	25.2	180.5	101	
B1	22.4	183.1	39	
B2	25.2	180.5	48	
B3	25.4	180.3	50	10.4
B4	25.4	180.3	50	5
B5	25.6	180.1	48	
B6	25.8	179.9	60	
B7	25.4	180.3	52	
B8	25.6	180.1	54	
B9	25.2	180.5	52	
B10	25.6	180.1	64	
B11	25.0	180.7	50	
B12	20.6	184.9	34	6
B13	20.0	185.4	36	
B14	20.2	185.2	43	
B15	20.8	184.7	47	

TABLE A-1 (cont'd)

**HWY 400/LANGSTAFF ROAD
SUMMARY OF PILE TIP CONDITIONS**

(Obtained From Pile Driving Records)

WEST PIER				
PILE I.D.	DRIVEN LENGTH (m)	TIP ELEV (m)	FINAL SET (Blows/200 mm)	RETAP (Blows/25 mm)
A1	21.6	184.1	40	8.9
A2	25.0	180.9	50	10
A3	24.8	181.1	55	6.7
A4	24.6	181.3	45	6.7
A5	25.0	180.9	38	6.7
A6	25.0	180.9	32	3.6
A7	26.0	179.9	28	10
A8	27.0	179.0	35	5.7
A9	26.4	179.6	25	5.7
A10	28.4	177.7	33	6.7
A11	30.2	175.9	34	10
A12	28.8	177.3	46	5
A13	32.8	173.5	35	2.5
B1	21.6	184.1	51	
B2	20.0	185.6	32	
B3	20.8	184.9	38	
B4	21.6	184.1	40	
B5	22.2	183.5	31	
B6	21.2	184.5	37	
B7	23.8	182.0	36	
B8	23.6	182.2	42	
B9	25.2	180.7	46	
B10	25.2	180.7	40	
B11	22.6	183.2	32	
B12	24.2	181.6	63	
B13	33.0	173.3	34	40

TABLE A-1

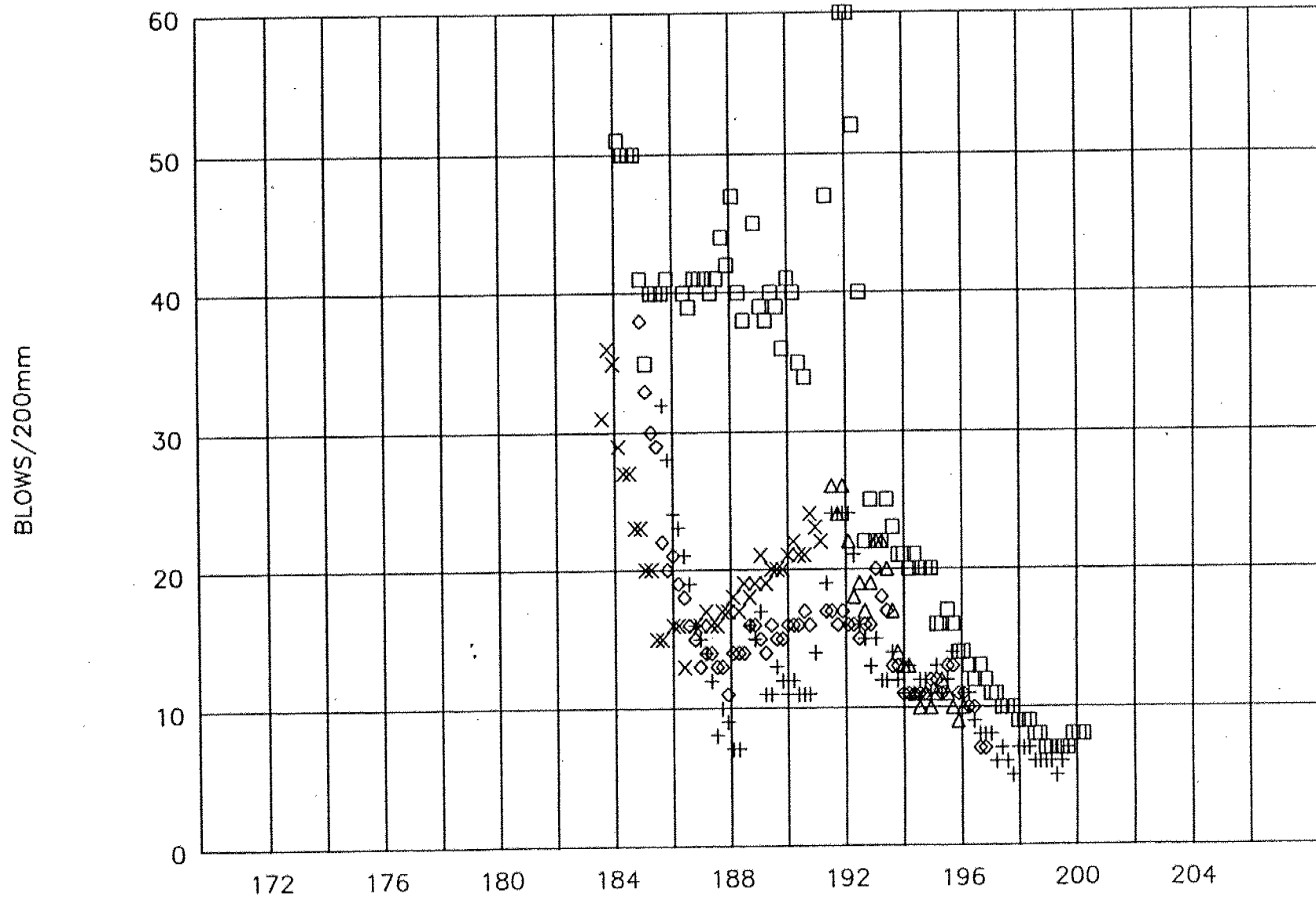
HWY 400/LANGSTAFF ROAD
SUMMARY OF PILE TIP CONDITIONS

(Obtained From Pile Driving Records)

WEST ABUTMENT									
PILE I.D.	DRIVEN LENGTH (m)	TIP ELEV (m)	FINAL SET (Blows/200 mm)	RETAP (Blows/25 mm)	PILE I.D.	DRIVEN LENGTH (m)	TIP ELEV (m)	FINAL SET (Blows/200 mm)	RETAP (Blows/25 mm)
A1	29.6	178.8	35		C1	19.8	188.6	40	
A2	23.0	185.6	33	41	C2	29.2	179.7	68	
A3	29.2	179.7	33		C3	21.8	186.7	34	
A4	28.6	179.8	26	5	C4	21.6	186.9	31	
A5	31.2	177.3	37		C5	26.6	182.2	22	6.3
A6	32	177.0	32	5	C6	29.6	179.3	54	
A7	32.8	176.3	33	5	C7	30.6	178.4	60	15.6
A8	33.6	175.0	31	6.9	C8	30.8	178.2	42	6.3
B1	25.8	182.9	35		C9	31.6	177.4	41	
B2	21.2	187.3	45		C10	32.8	176.3	36	
B3	20.2	188.2	39		C11	34.0	175.1	39	
B4	22.6	186.0	32	8.3	C12	33.8	175.3	34	
B5	24.6	184.1	35	8.3	C13	36.6	172.7	47	
B6	29.6	179.3	40	5.2	C14	33.6	175.5	36	
B7	30.2	178.7	50	8.3					
B8	29.4	179.5	36	8.3					
B9	31.4	177.6	33	8.9					
B10	32.6	176.5	20	25					
B11	37.8	171.5	37	25					
B12	34.4	174.8	30	7.8					
B13	32.6	176.5	32	5.7					
B14	33.6	175.5	33	5					
B15	21.6	182.5	29						
B16	22.0	182.9	29						

HWY 400/LANGSTAFF RD.

WEST PIER

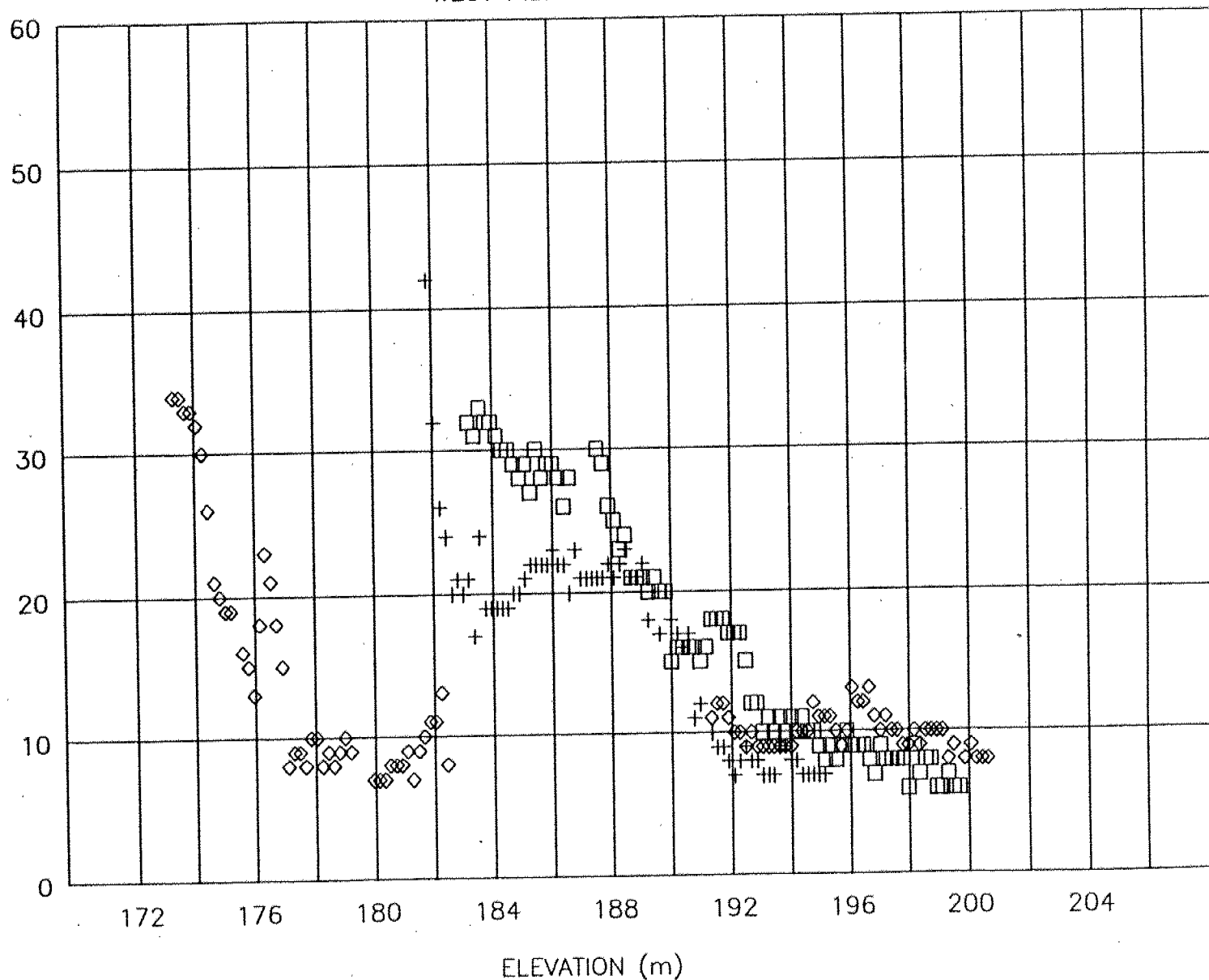


PILE B1
 PILE B2
 PILE B3
 PILE B4
 PILE B5

HWY 400/LANGSTAFF RD.

WEST PIER - EAST SIDE

BLOWS/200mm



□ PILE B11 + PILE B12 ◇ PILE B13

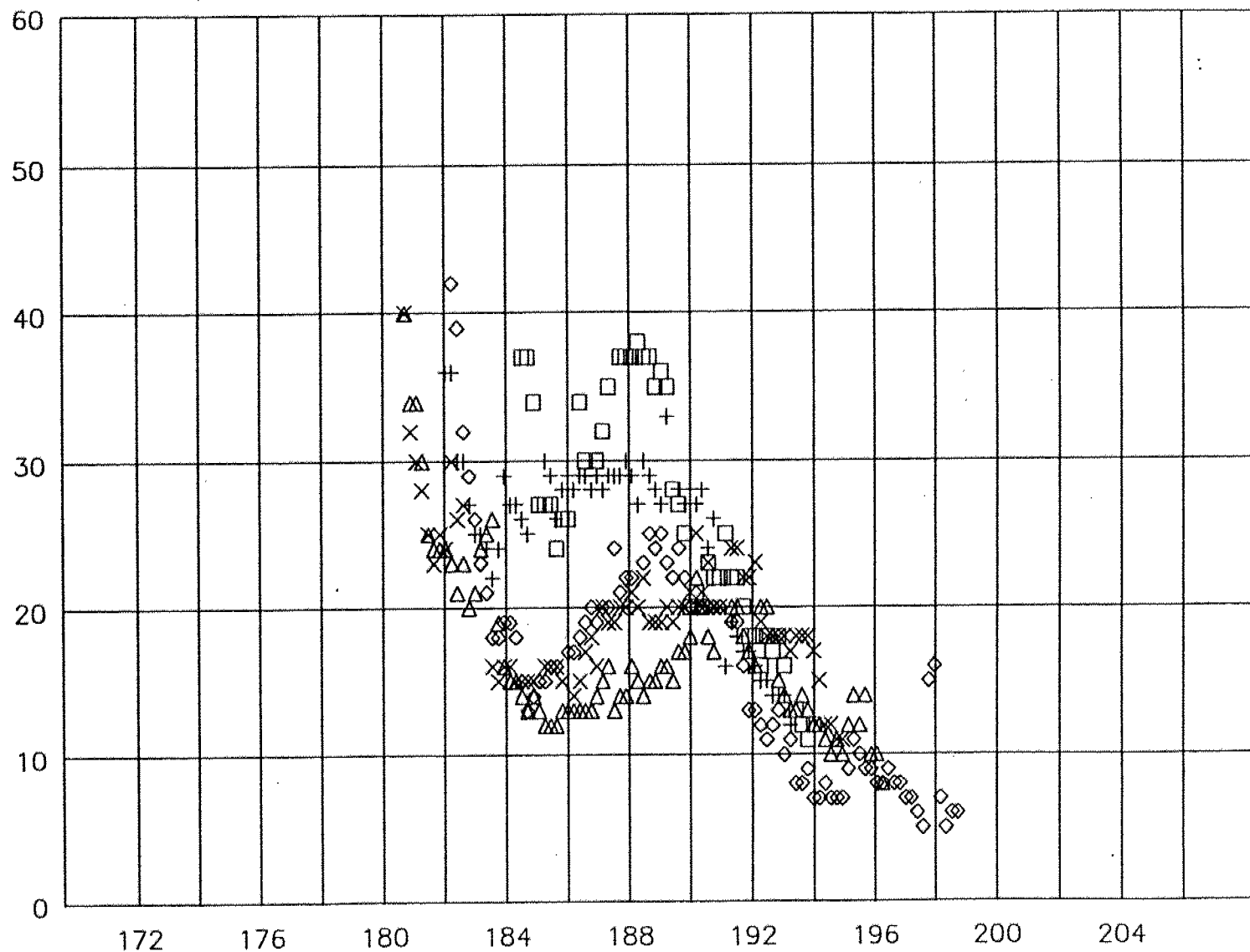
901-1404

HWY 400/LANGSTAFF RD.

WEST PIER

BLOWS/200mm

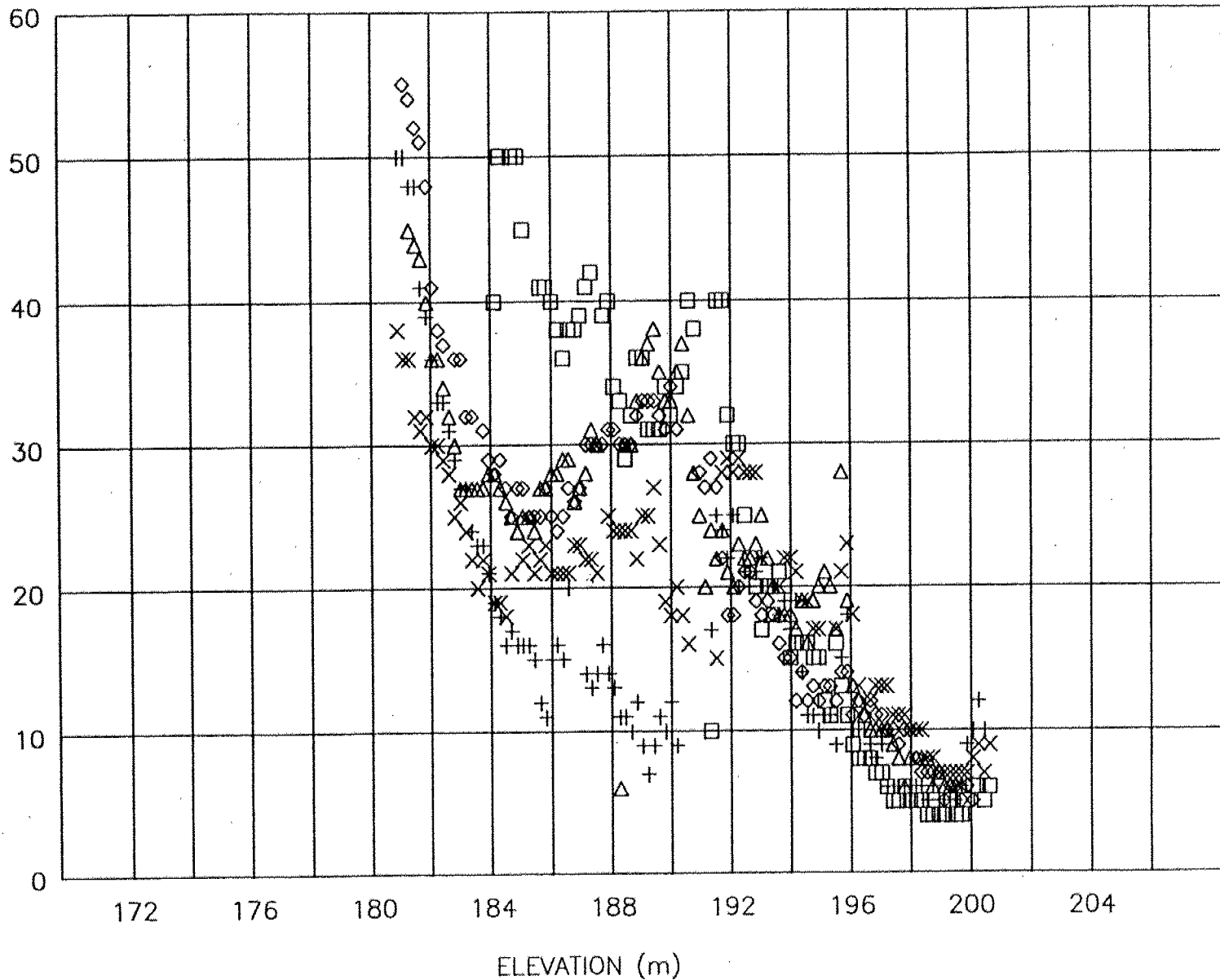
Golder Associates



□ PILE B6 + PILE B7 ◇ PILE B8 △ PILE B9 × PILE B10

HWY 400/LANGSTAFF RD.

WEST PIER

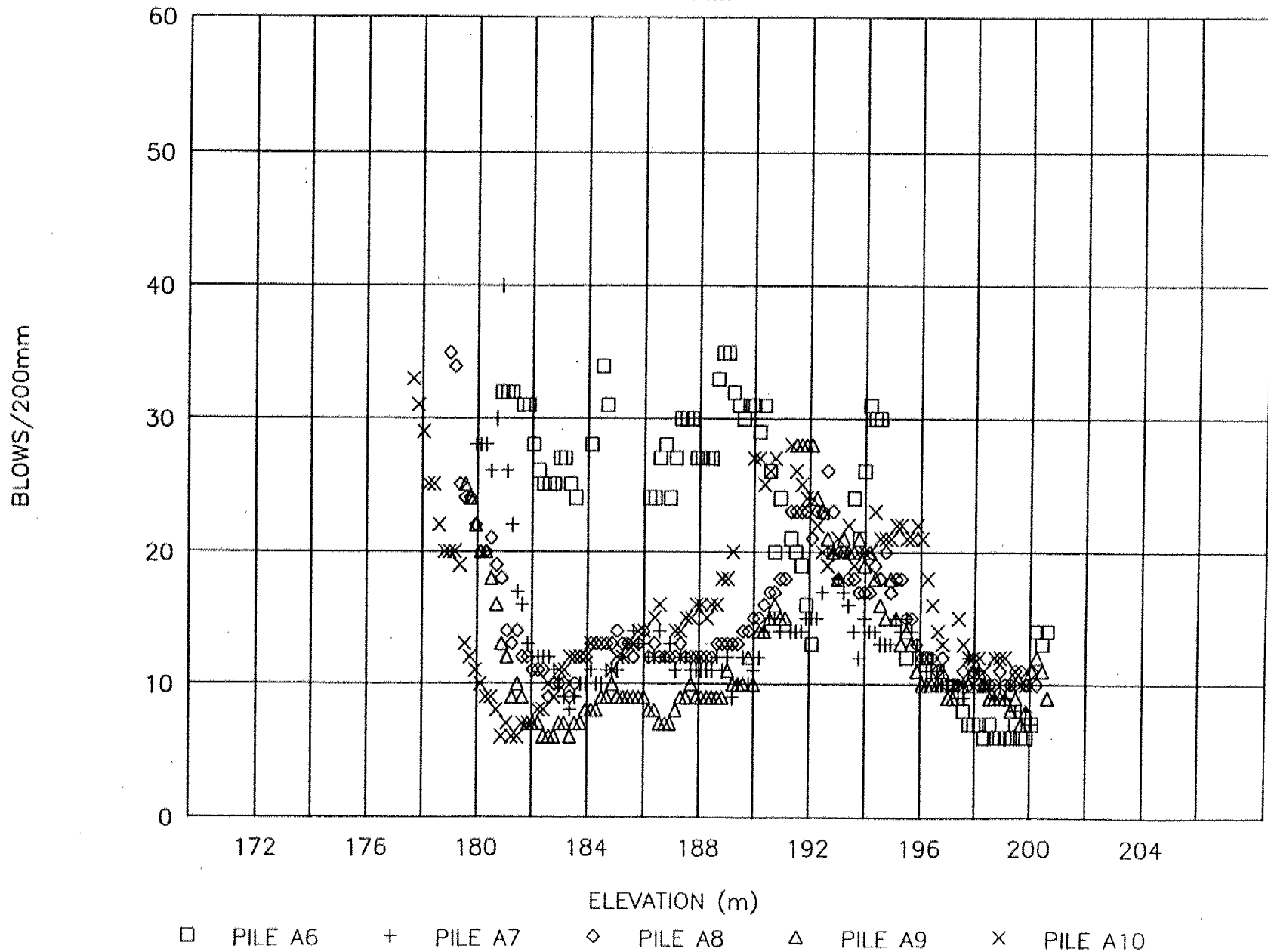


□ PILE A1 + PILE A2 ◇ PILE A3 △ PILE A4 × PILE A5

901-1404

HWY 400/LANGSTAFF RD.

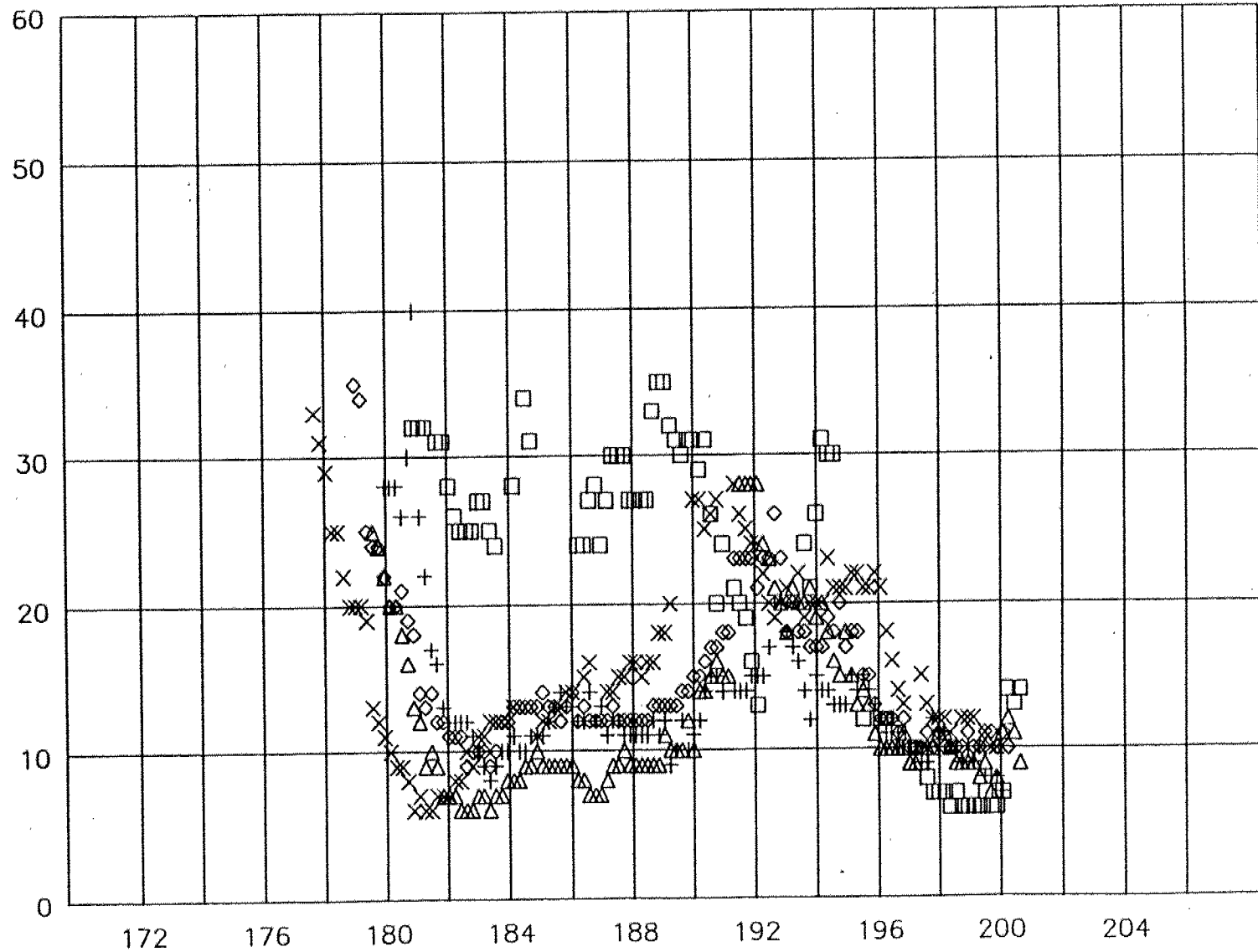
WEST PIER



HWY 400/LANGSTAFF RD.

WEST PIER

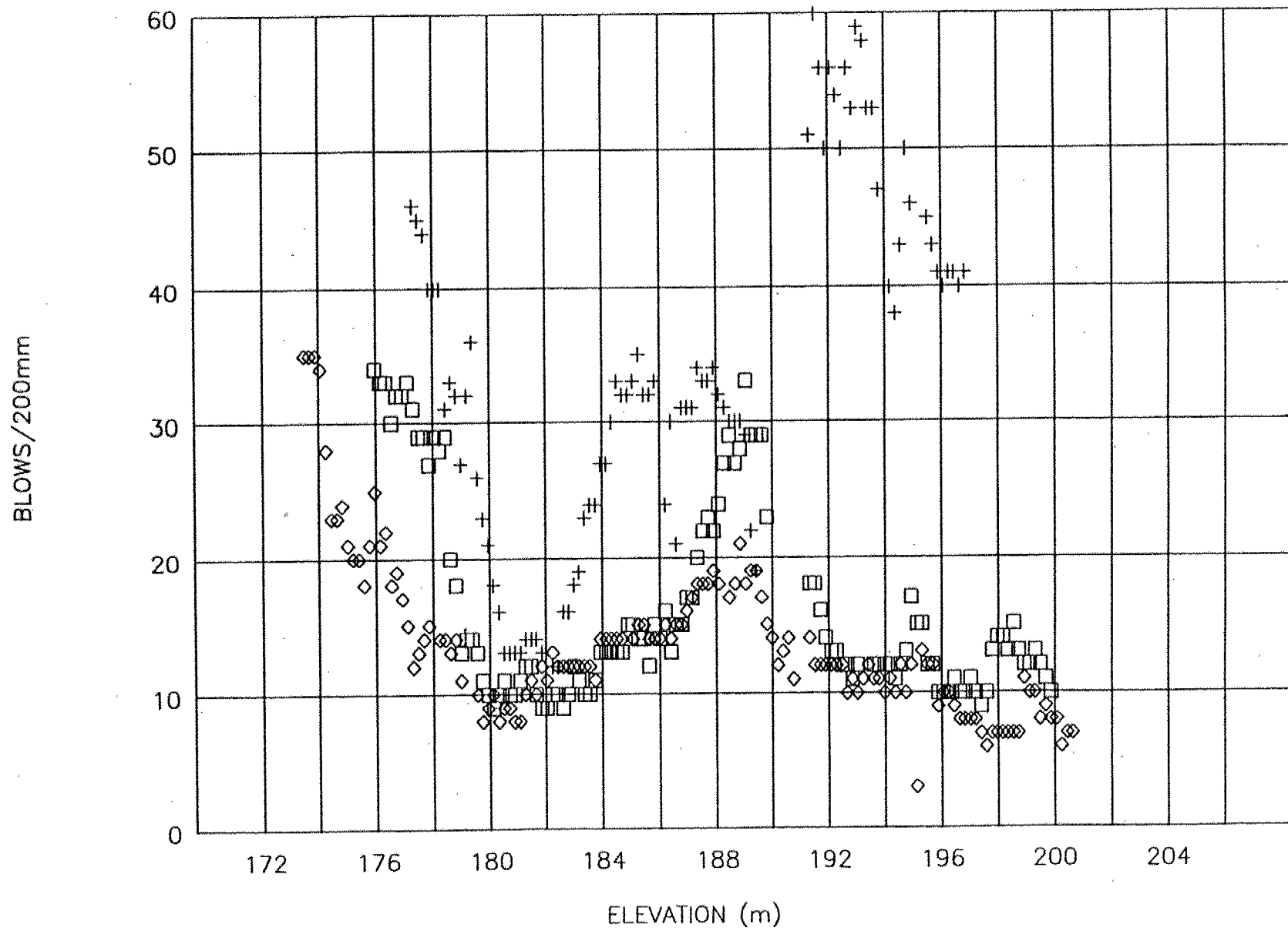
BLOWS/200mm



□ PILE A6 + PILE A7 ◇ PILE A8 △ PILE A9 × PILE A10

HWY 400/LANGSTAFF RD.

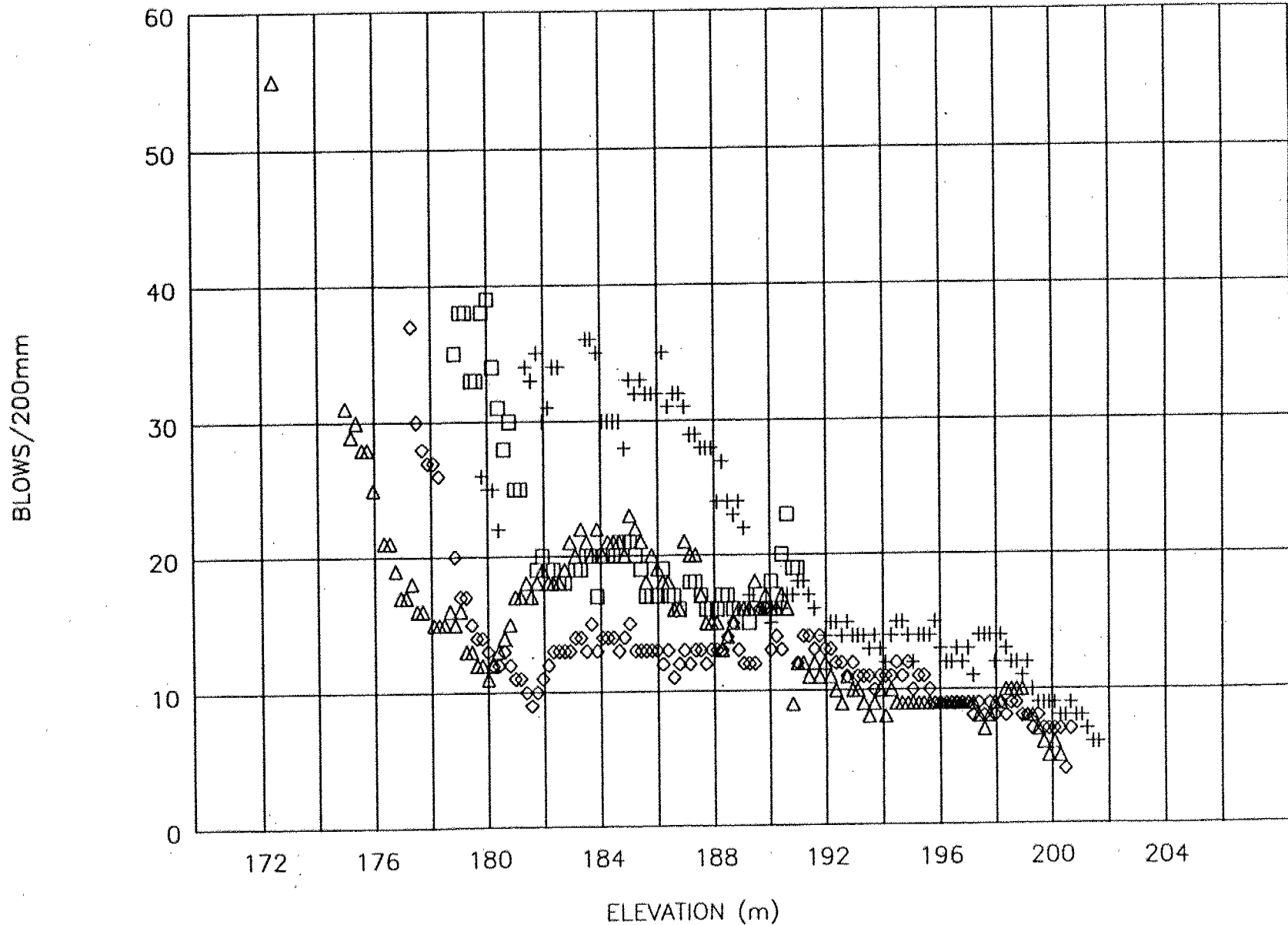
WEST PIER



□ PILE A11 + PILE A12 ◇ PILE A13

HWY 400/LANGSTAFF RD.

WEST ABUTMENT

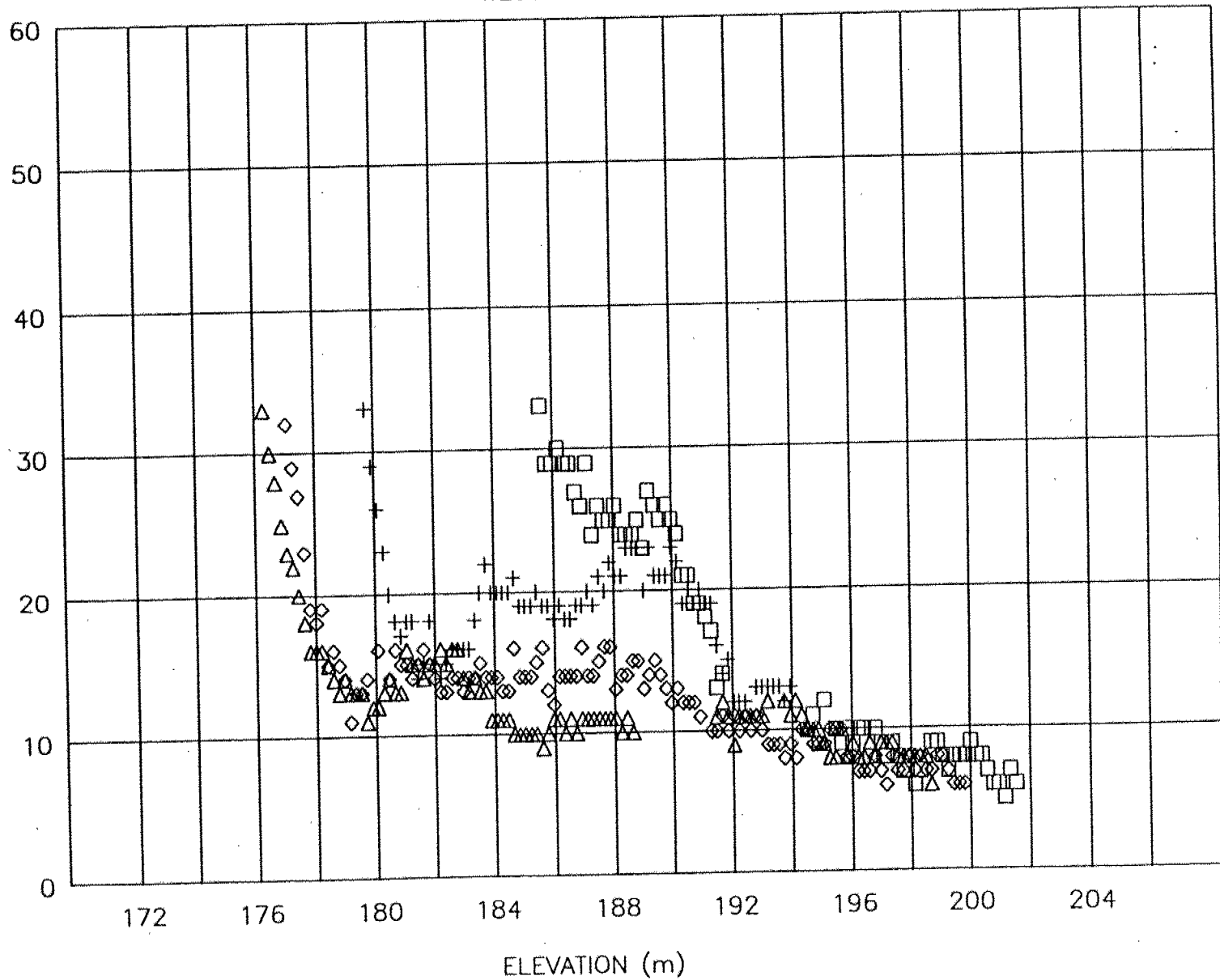


□ PILE A1 + PILE A4 ◇ PILE A5 △ PILE A8

HWY 400/LANGSTAFF RD.

WEST ABUTMENT

BLOWS/200mm

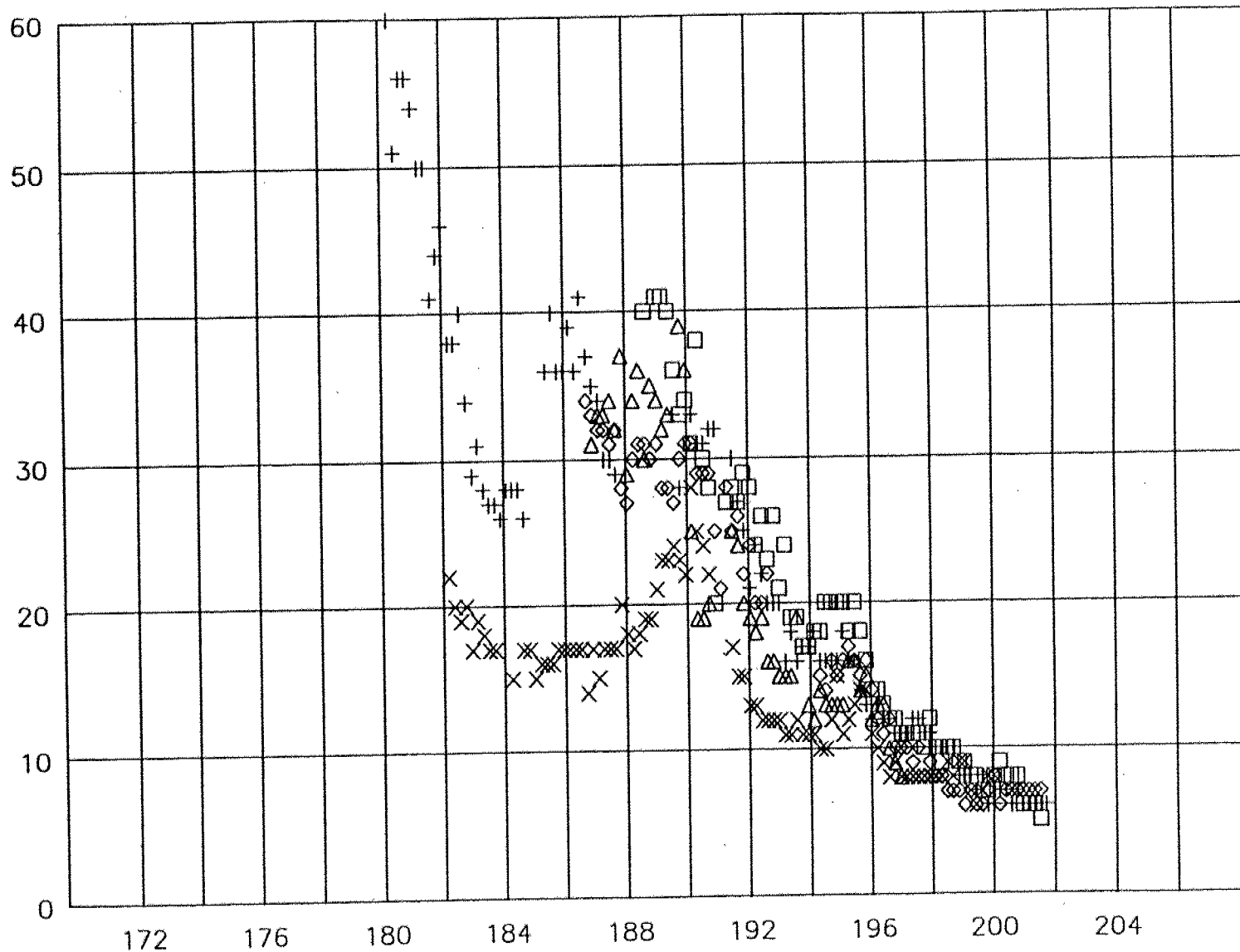


□ PILE A2 + PILE A3 ◇ PILE A6 △ PILE A7

HWY 400/LANGSTAFF RD.

WEST ABUTMENT

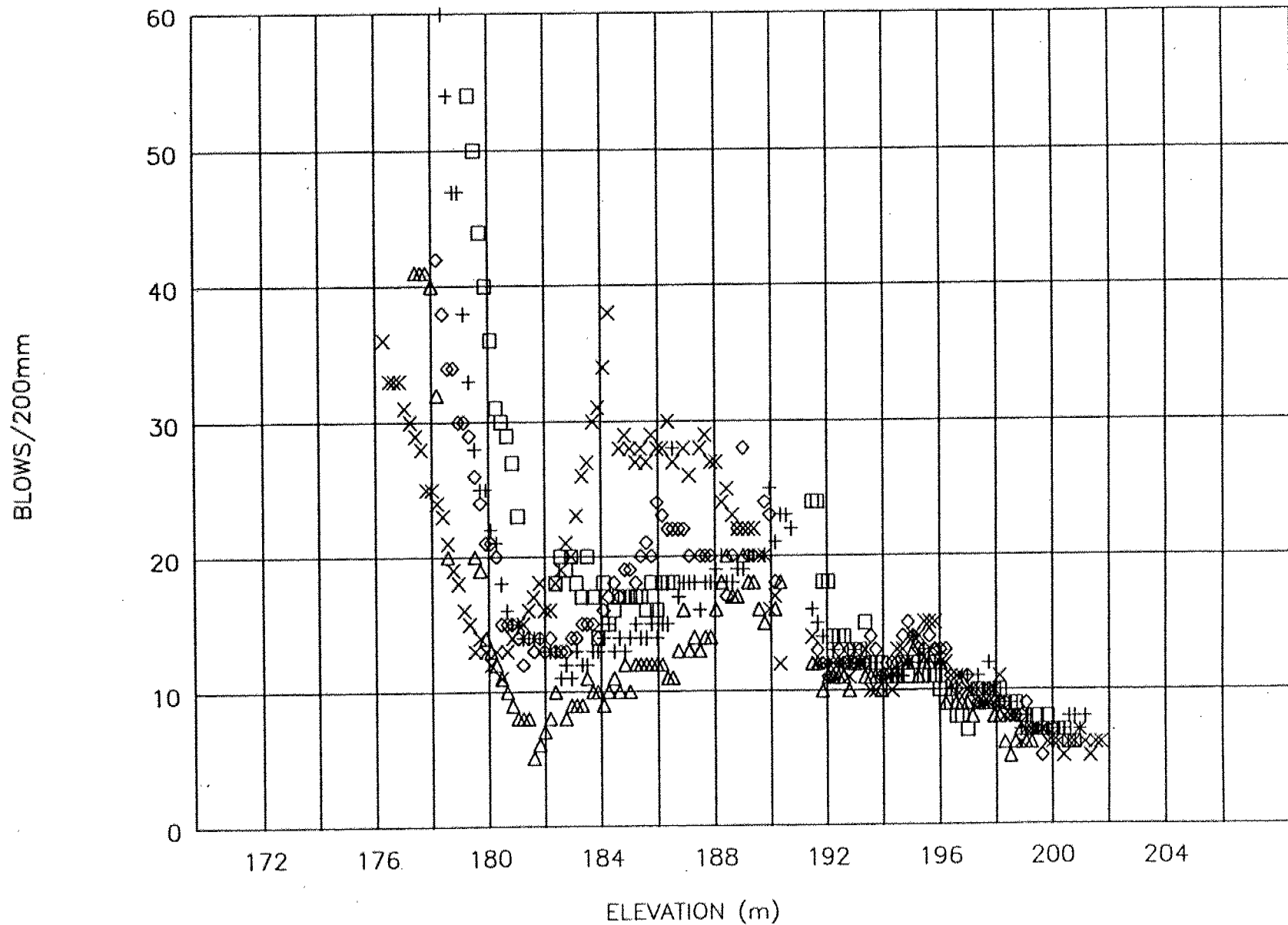
BLOWS/200mm



□ PILE C1 + PILE C2 ◇ PILE C3 △ PILE C4 × PILE C5

HWY 400/LANGSTAFF RD.

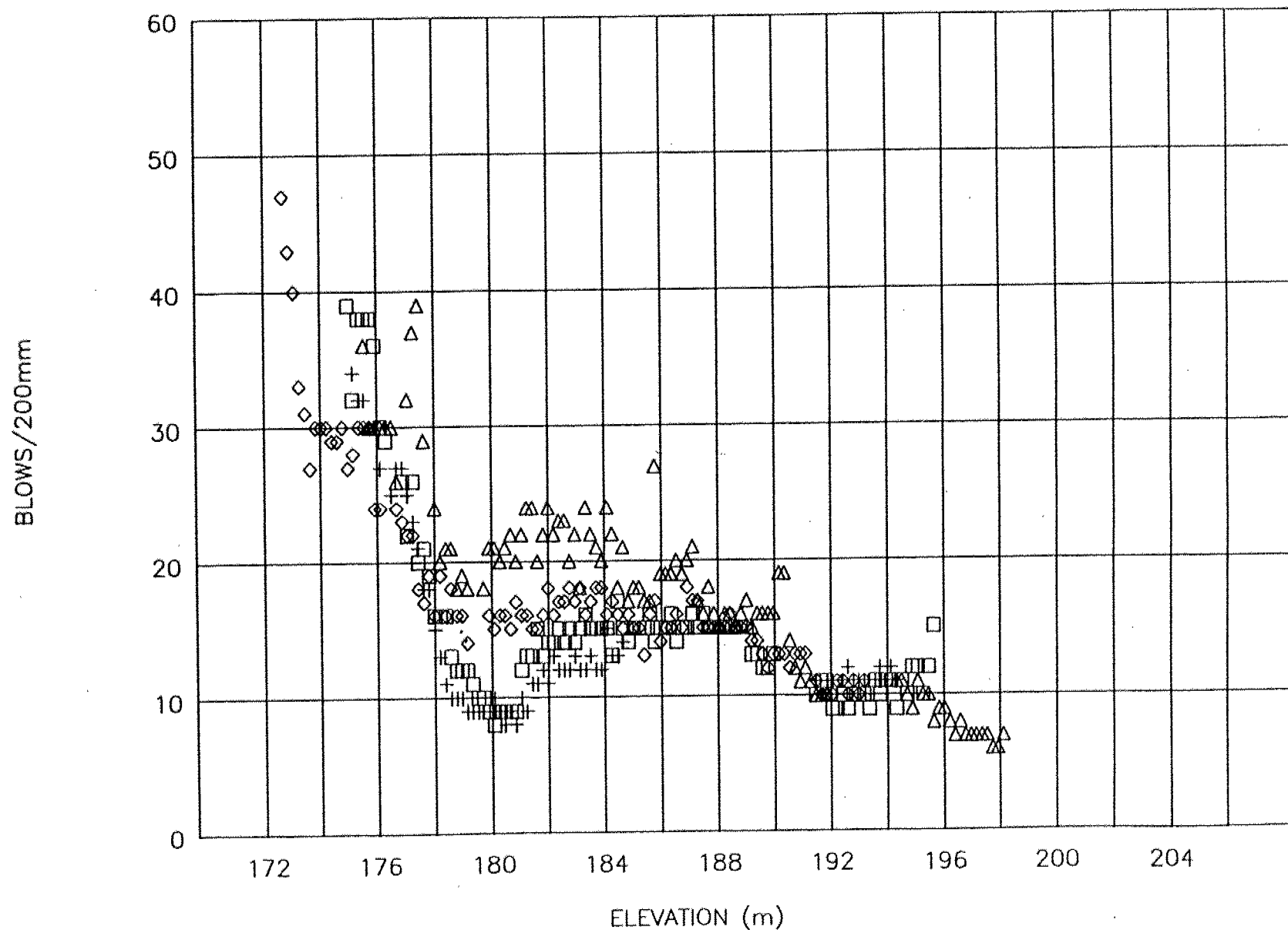
WEST ABUTMENT



□ PILE C6 + PILE C7 ◇ PILE C8 △ PILE C9 × PILE C10

HWY 400/LANGSTAFF RD.

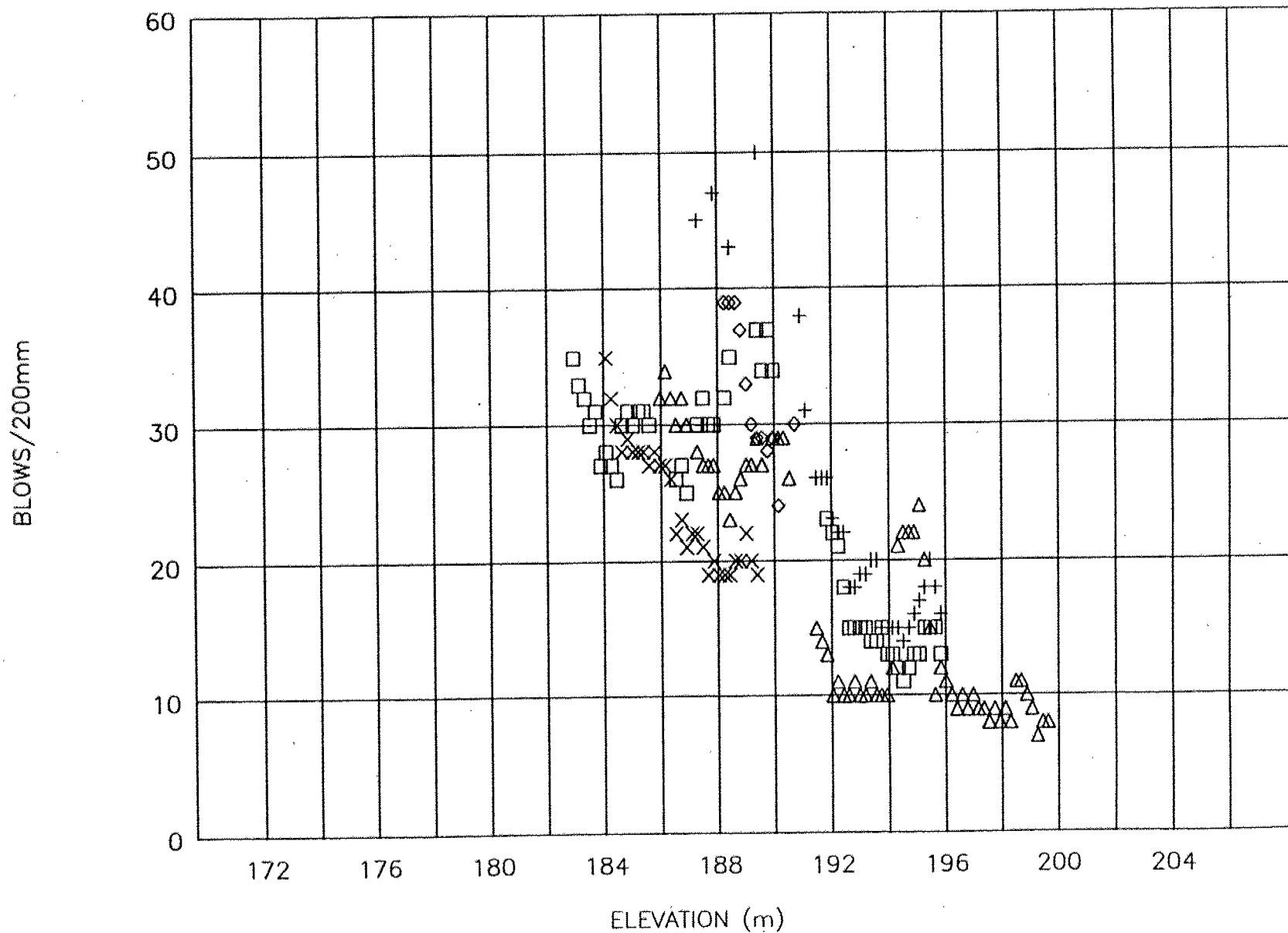
WEST ABUTMENT



□ PILE C11 + PILE C12 ◇ PILE C13 Δ PILE C14

HWY 400/LANGSTAFF RD.

WEST ABUTMENT

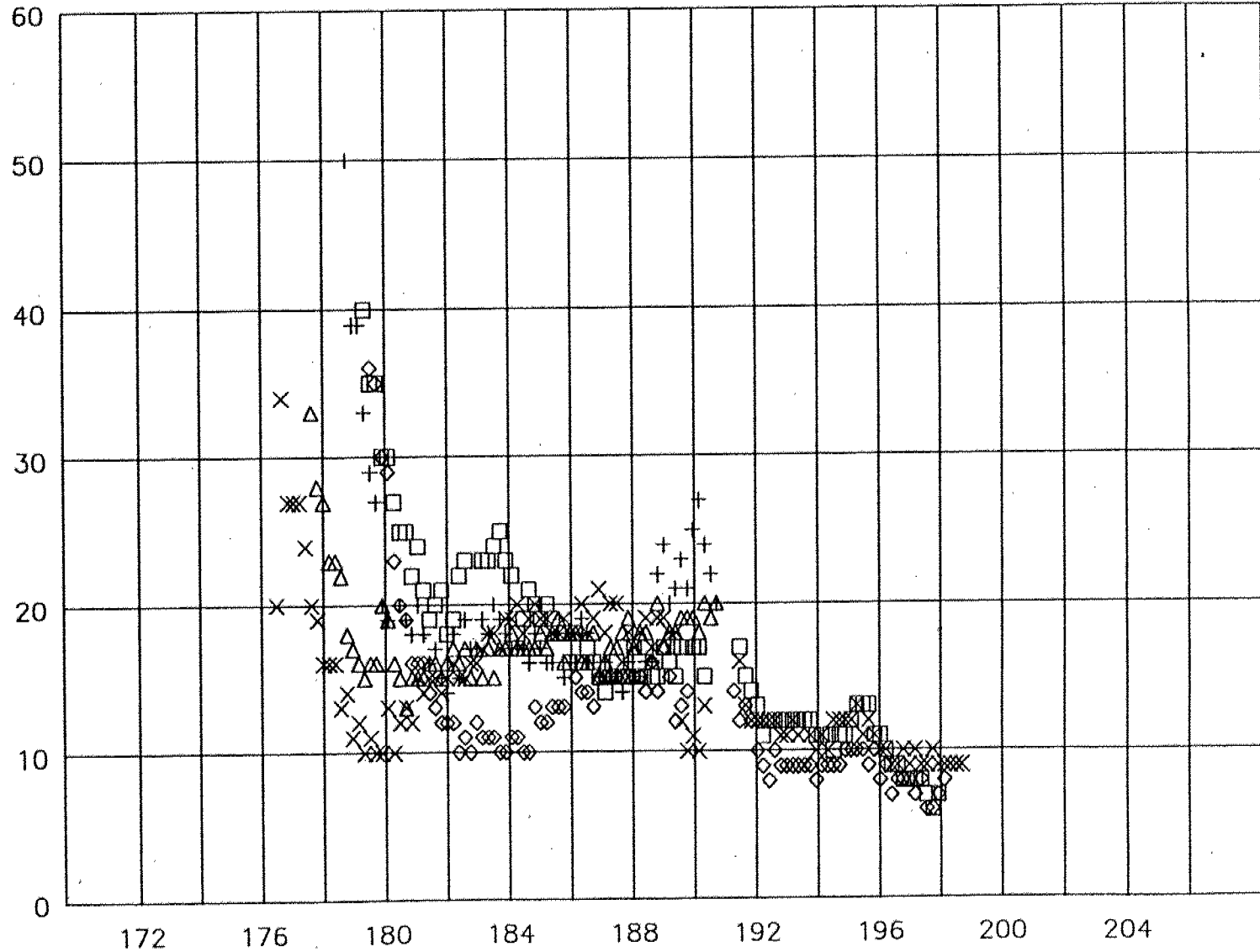


□ PILE B1 + PILE B2 ◇ PILE B3 △ PILE B4 × PILE B5

HWY 400/LANGSTAFF RD.

WEST ABUTMENT

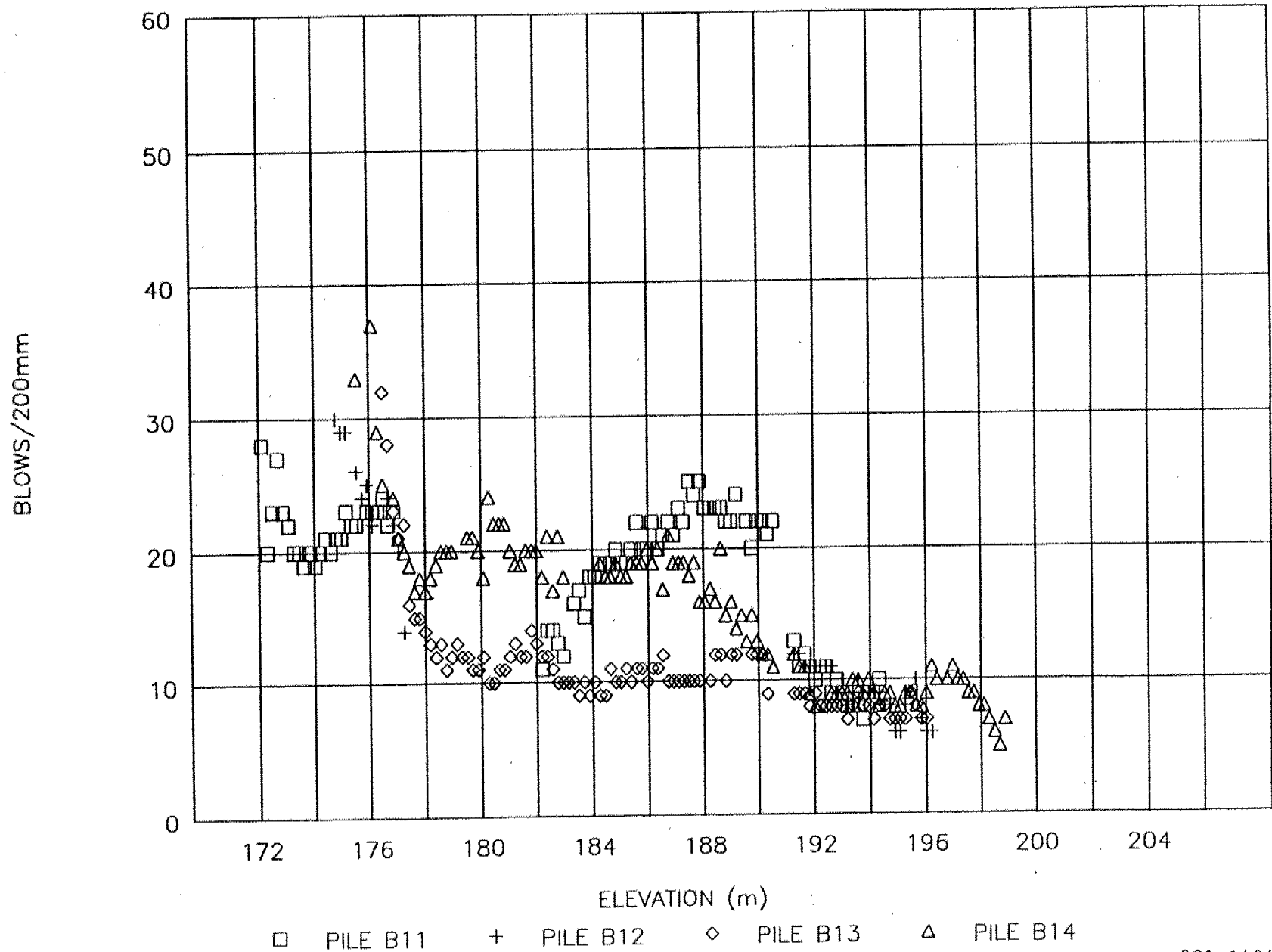
BLOWS/200mm



□ PILE B6 + PILE B7 ◇ PILE B8 △ PILE B9 × PILE B10

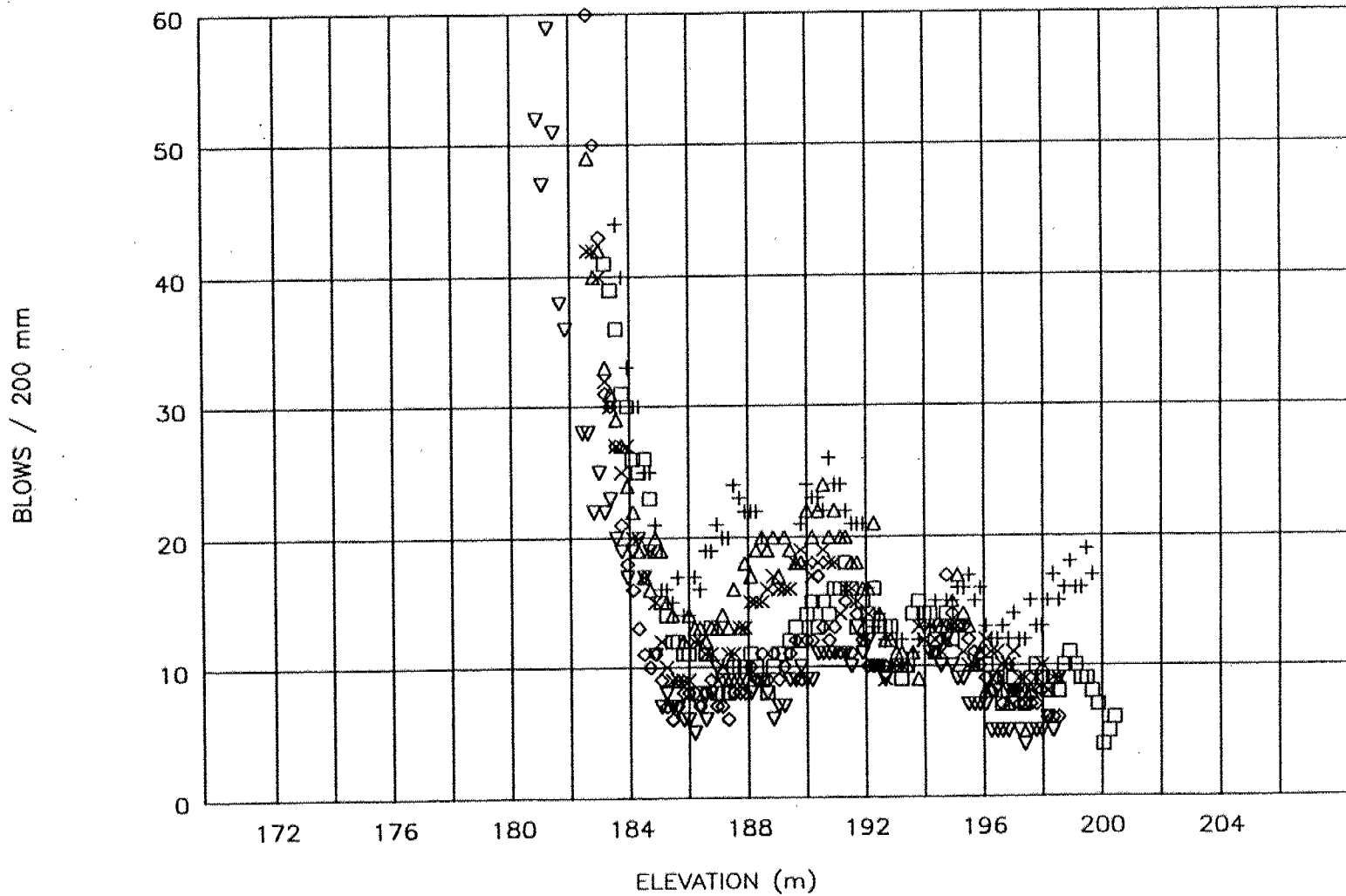
HWY 400/LANGSTAFF RD.

WEST ABUTMENT



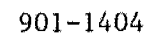
HWY 400 / LANGSTAFF RD.

EAST PIER

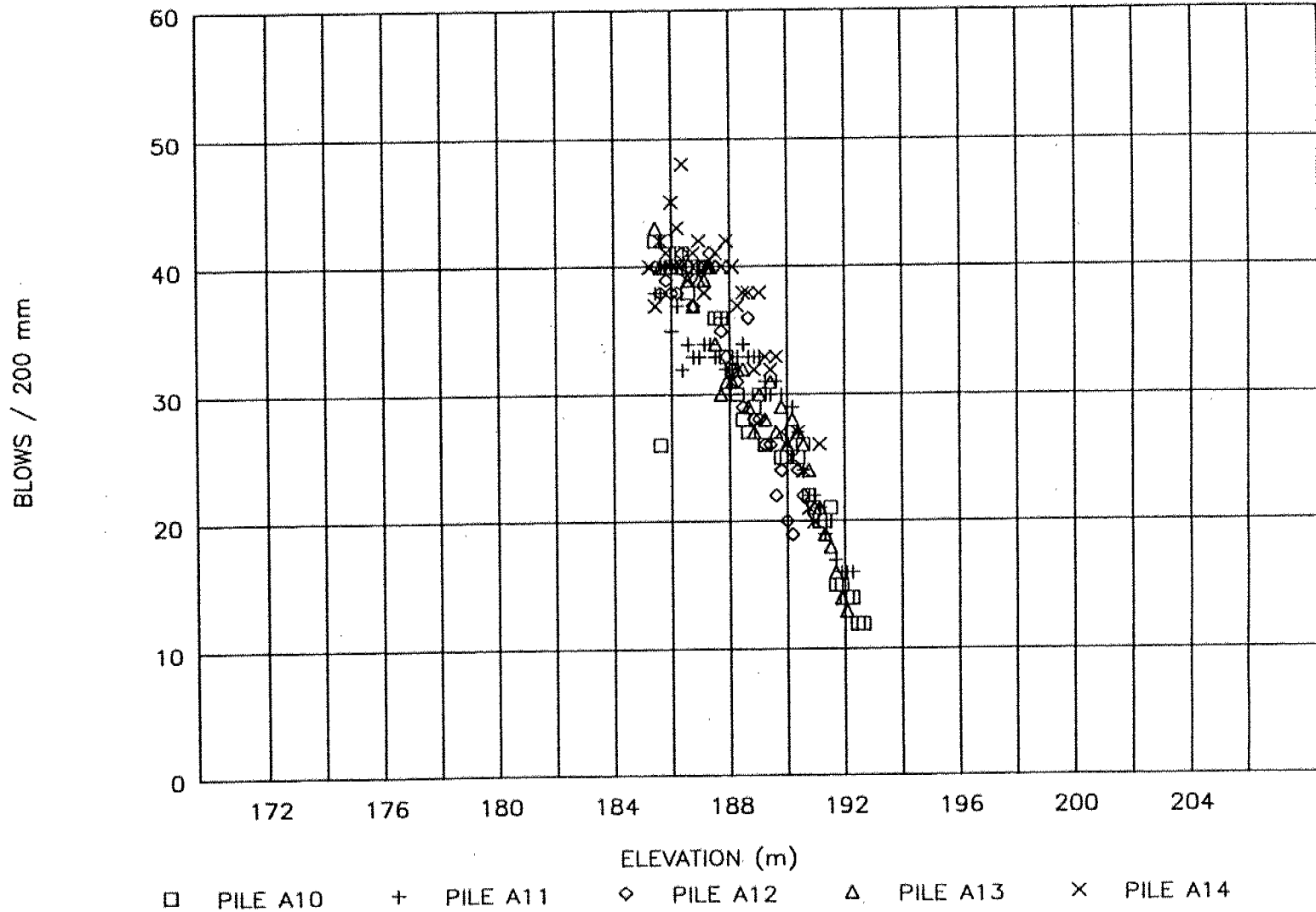


PILE A1
 PILE A2
 PILE A3
 PILE A4
 PILE A5
 PILE A6

Golder Associates



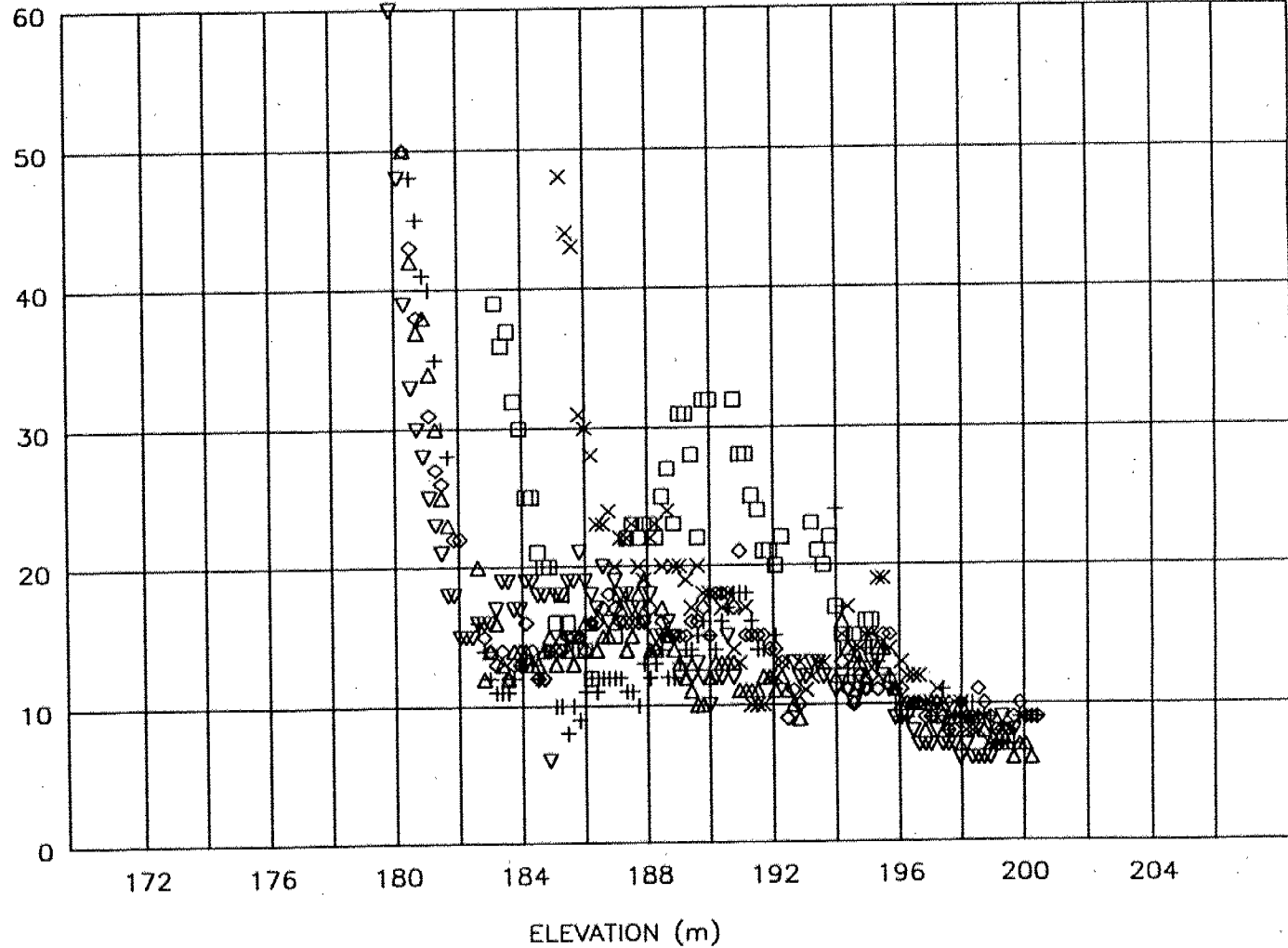
HWY 400 / LANGSTAFF RD. EAST PIER



HWY 400 / LANGSTAFF RD.

EAST PIER

BLOWS / 200 mm

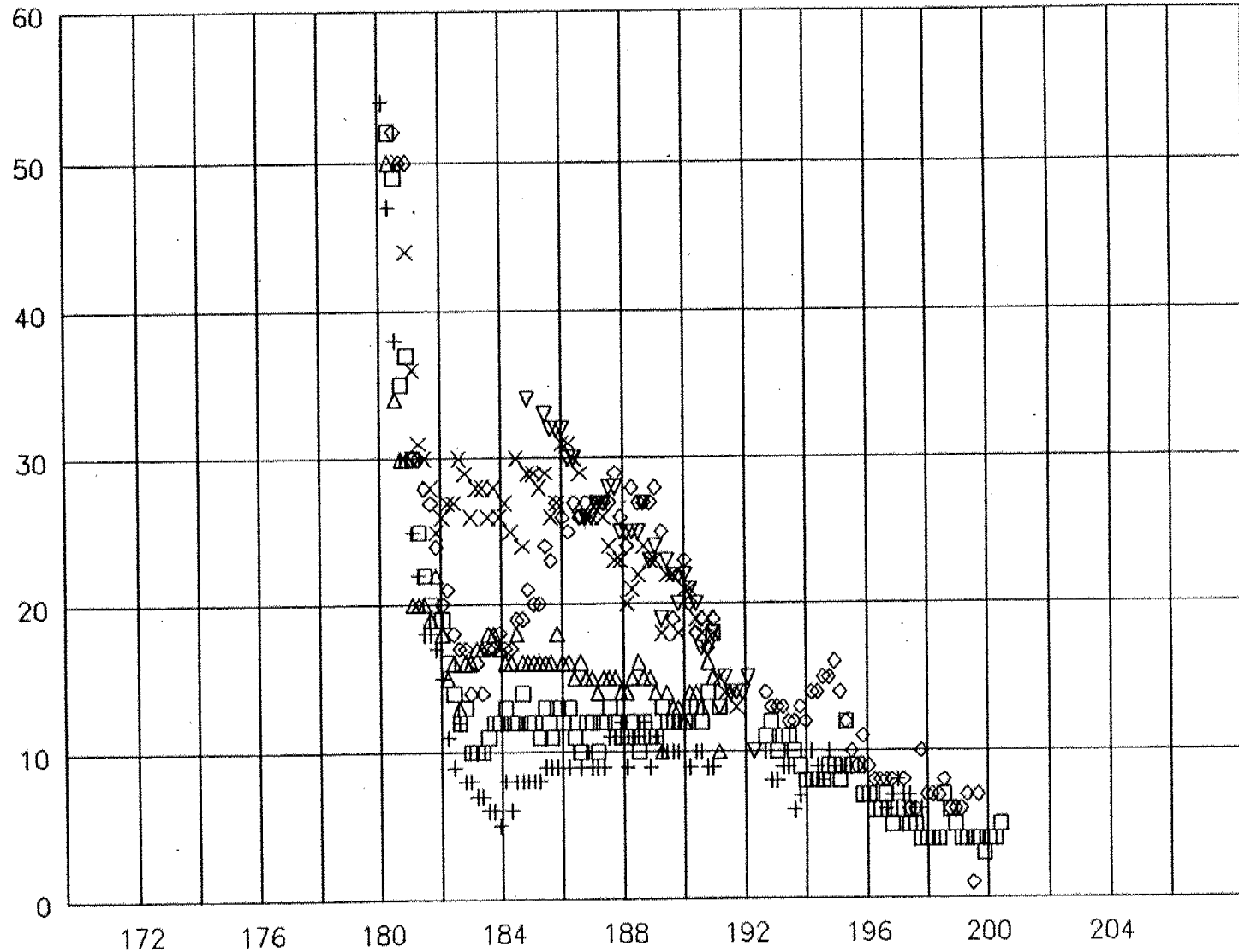


□ PILE B1 + PILE B2 ◇ PILE B3 Δ PILE B4 × PILE B5 ▽ PILE B6

HWY 400 / LANGSTAFF RD.

EAST PIER

BLOWS / 200 mm



□ PILE B7

+ PILE B8

◇ PILE B9

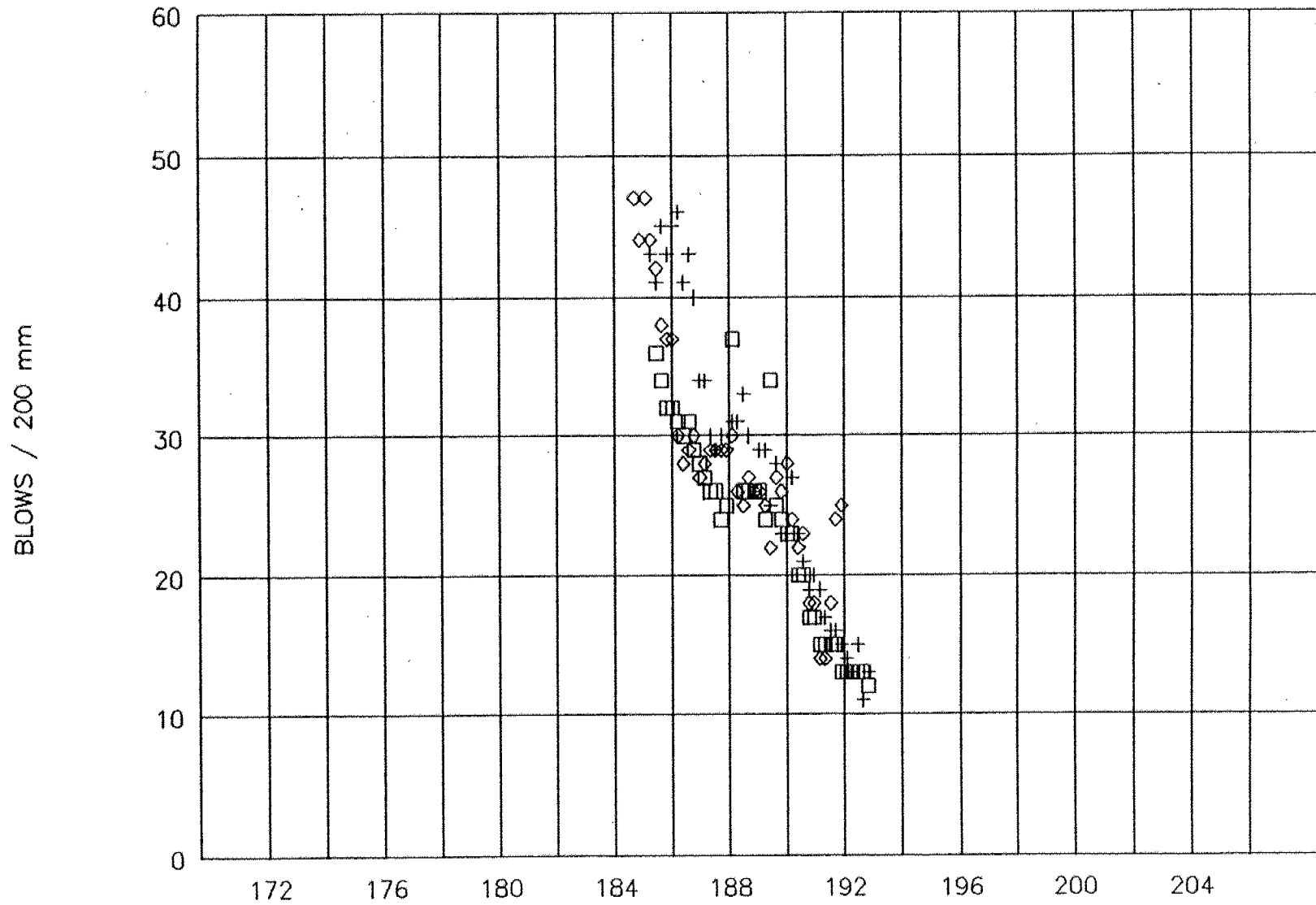
△ PILE B10

× PILE B11

▽ PILE B12

HWY 400 / LANGSTAFF RD.

EAST PIER

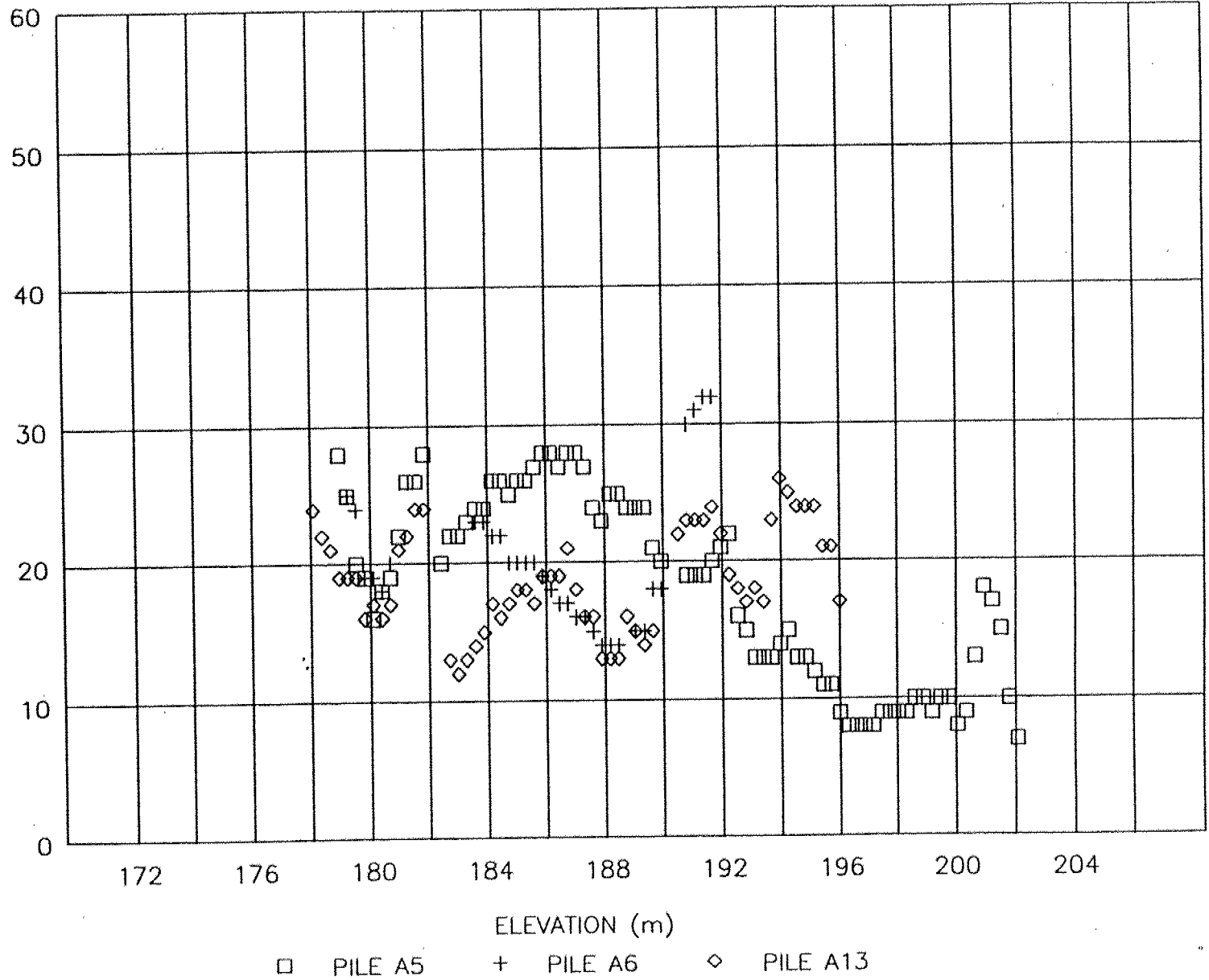


□ PILE B13 + PILE B14 ◇ PILE B15

HWY 400 / LANGSTAFF RD. EAST ABUTMENT

Golder Associates

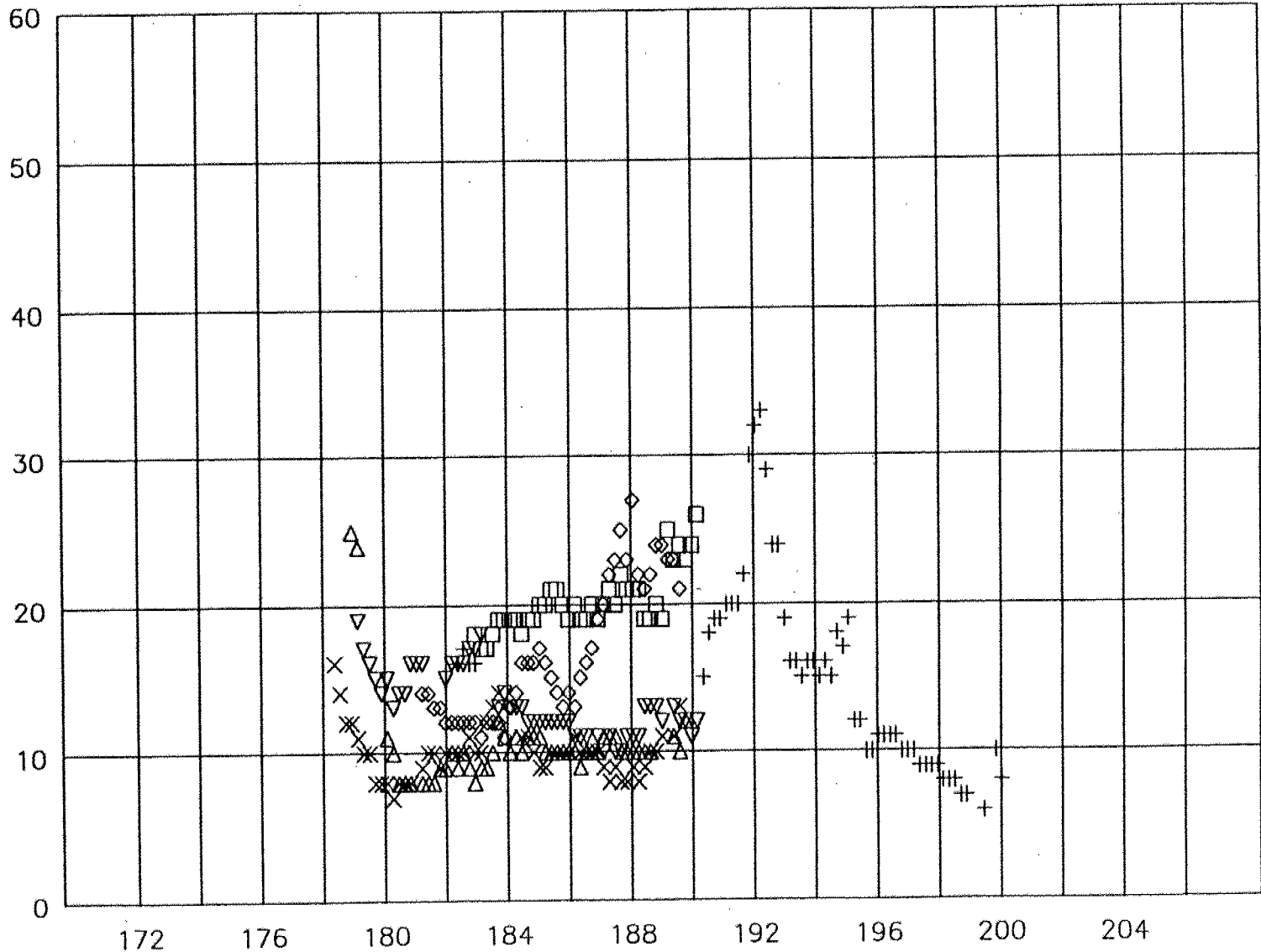
BLOWS / 305 mm



HWY 400 / LANGSTAFF RD.

EAST ABUTMENT

Blows / 200 mm

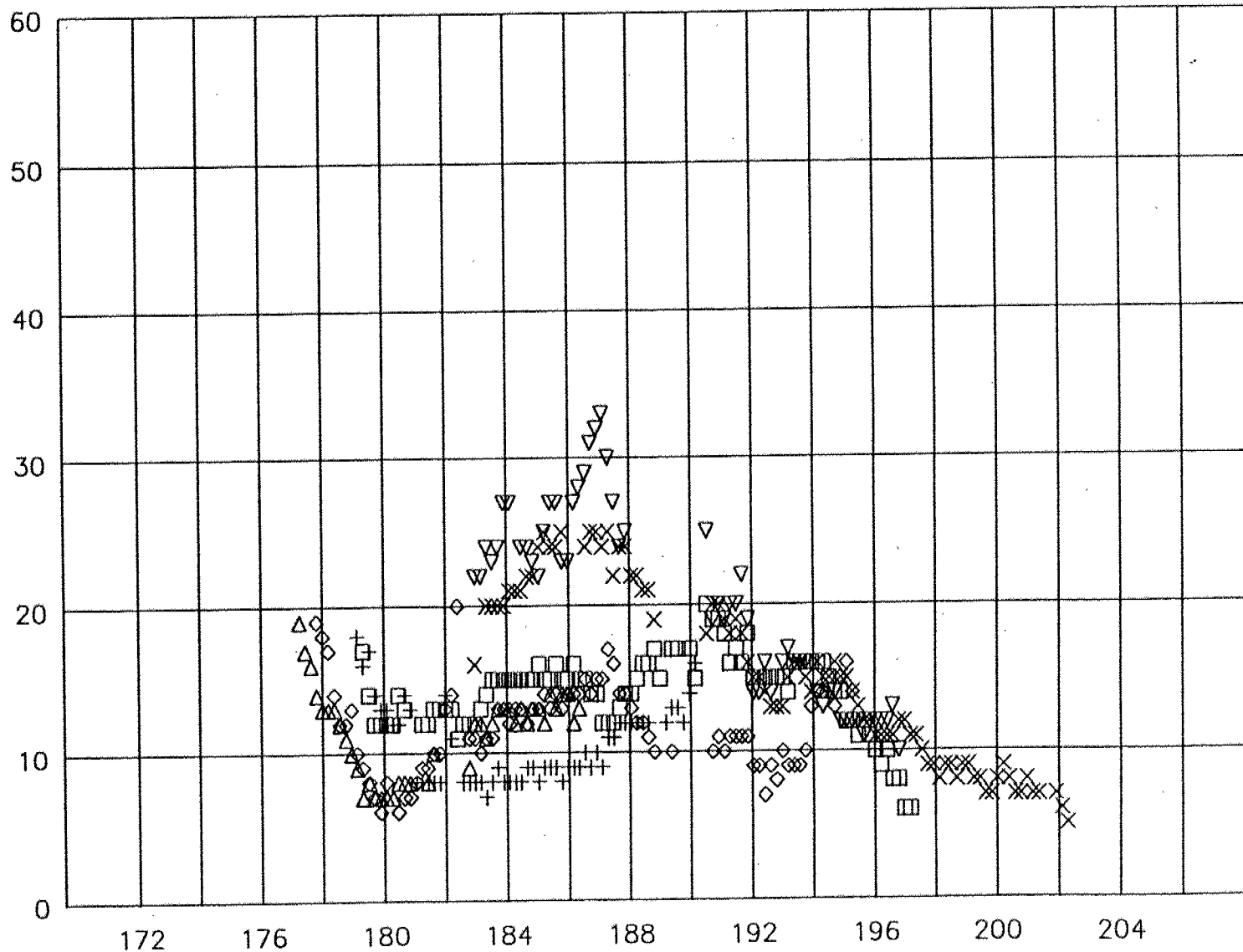


PILE A1
 PILE A2
 PILE A3
 PILE A4
 PILE A7
 PILE A8

HWY 400 / LANGSTAFF RD.

EAST ABUTMENT

Blows / 200 mm



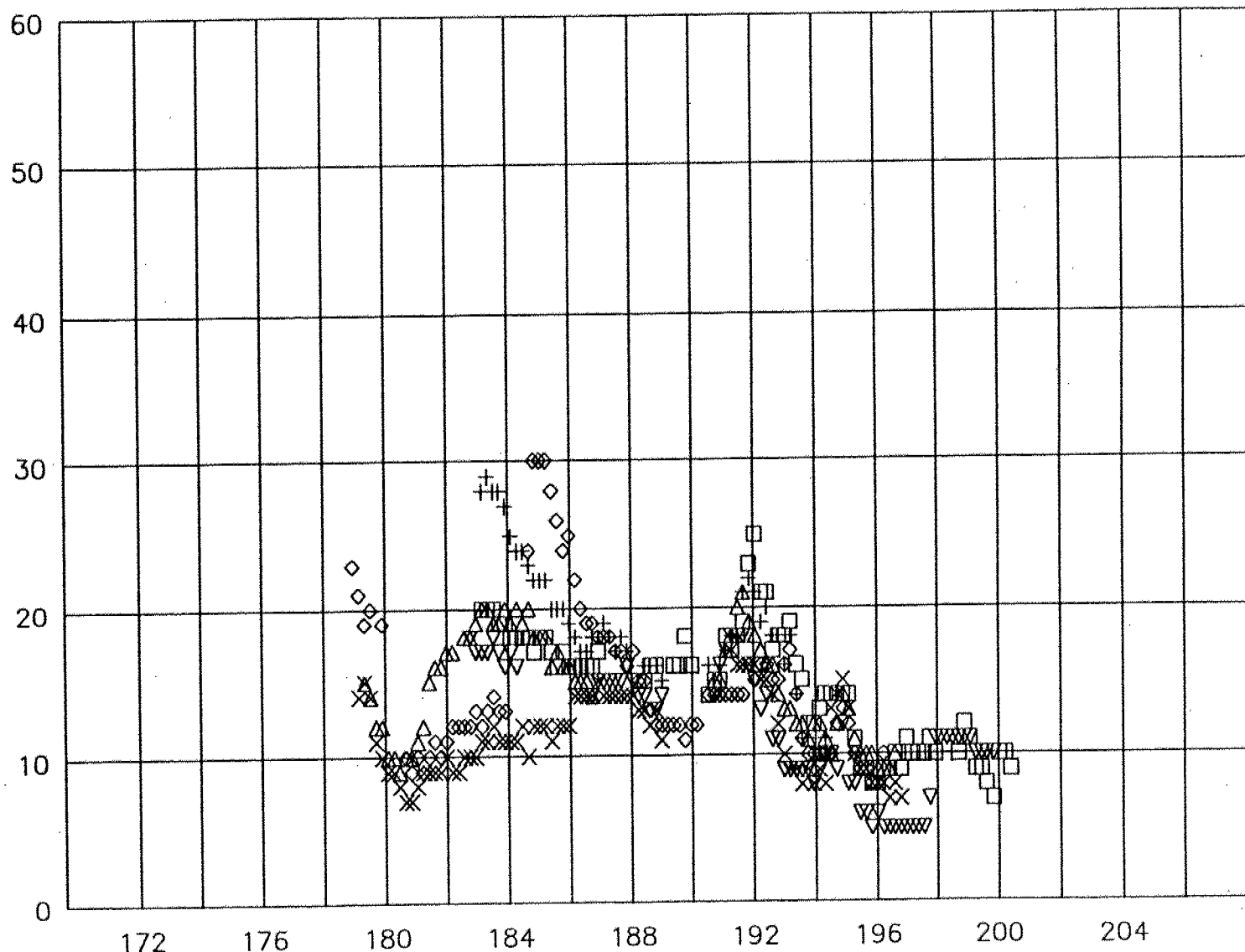
PILE A9
 PILE A10
 PILE A11
 PILE A12
 PILE A15
 PILE A16

901-1404

HWY 400 / LANGSTAFF RD.

EAST ABUTMENT

BLOWS/200mm

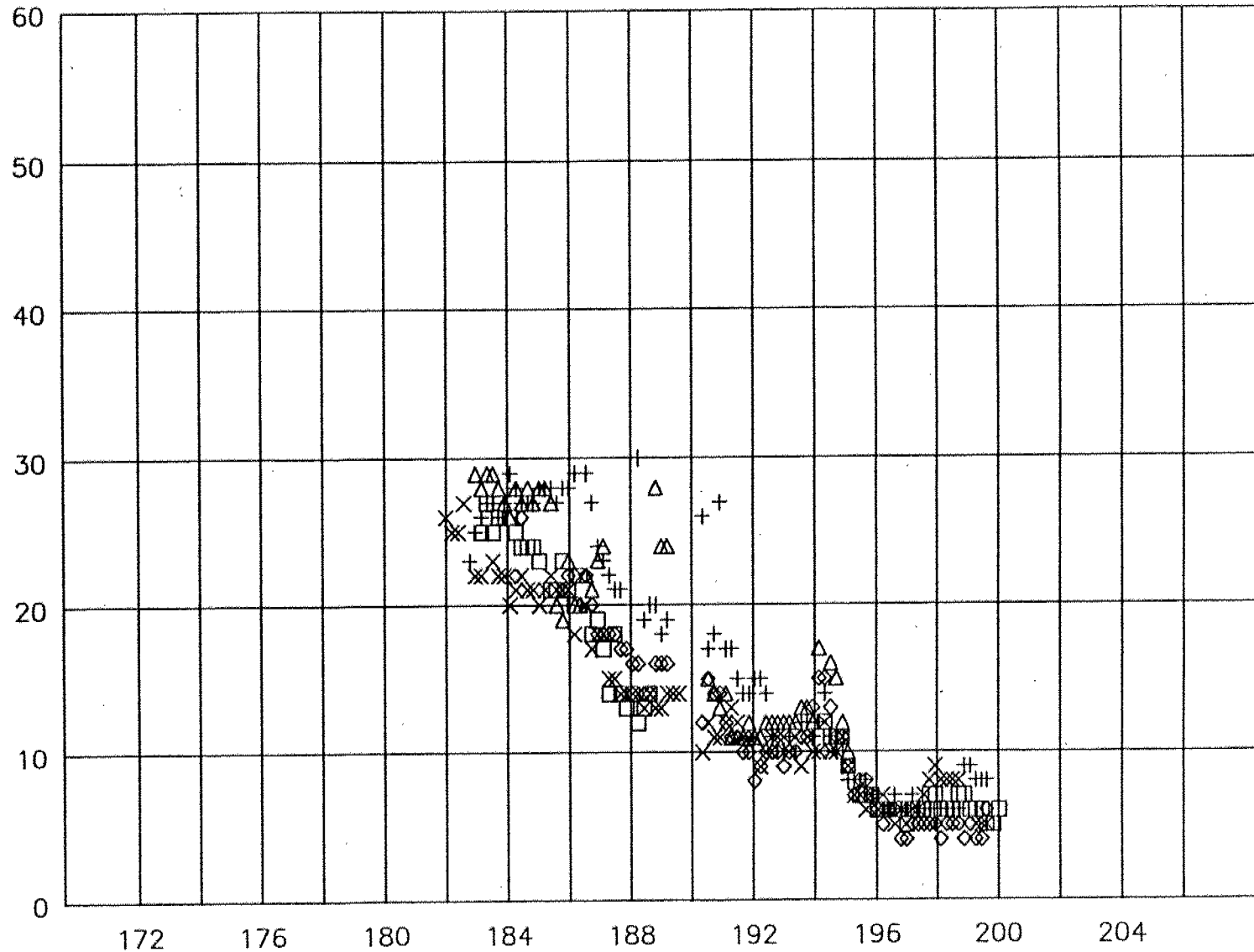


□ PILE B1 + PILE B2 ◇ PILE B3 △ PILE B4 × PILE B5 ▽ PILE B6

HWY 400 / LANGSTAFF RD.

EAST ABUTMENT

BLOWS/200mm

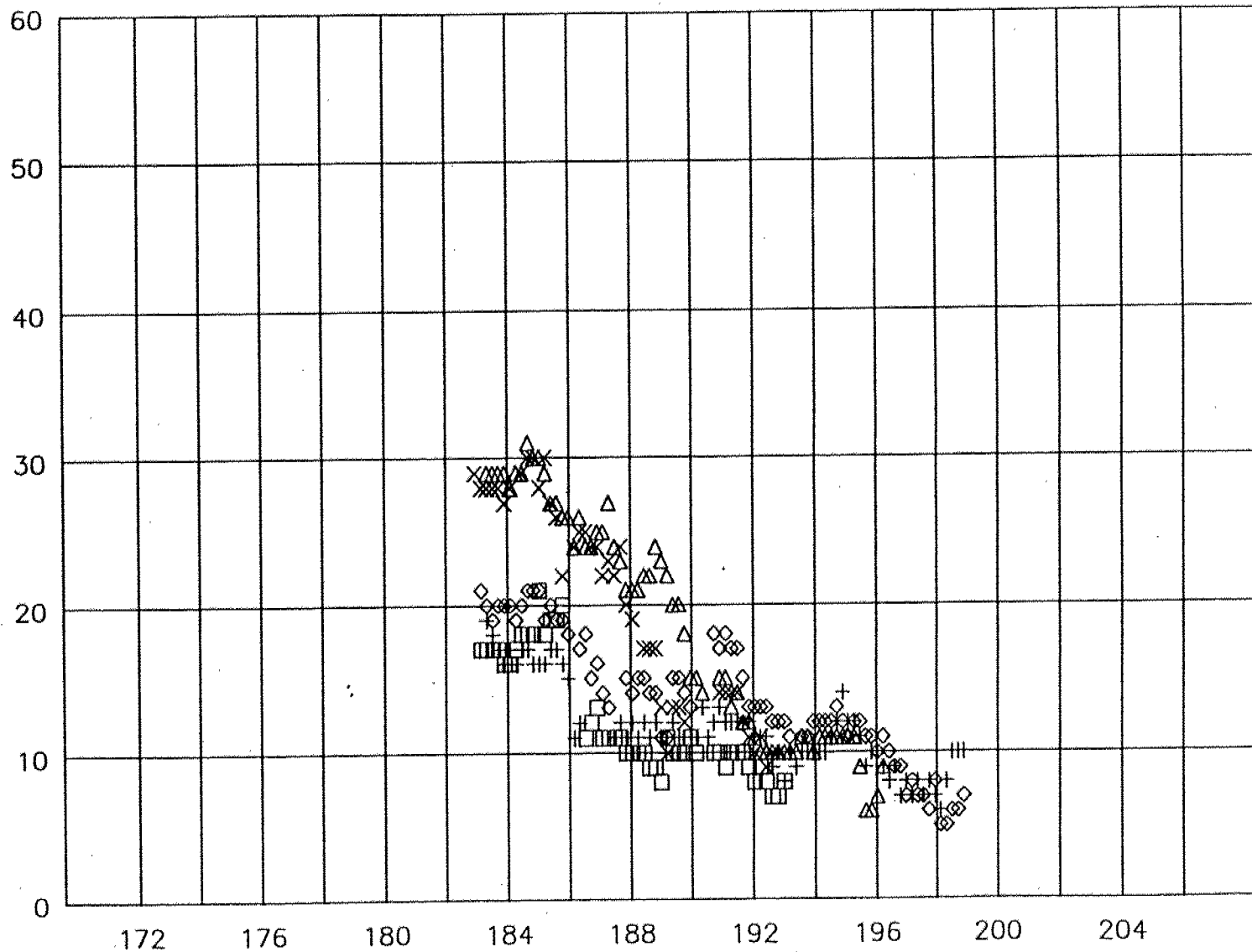


□ PILE B7 + PILE B8 ◇ PILE B9 △ PILE B10 × PILE B11

HWY 400 / LANGSTAFF RD.

EAST ABUTMENT

BLOWS/200mm

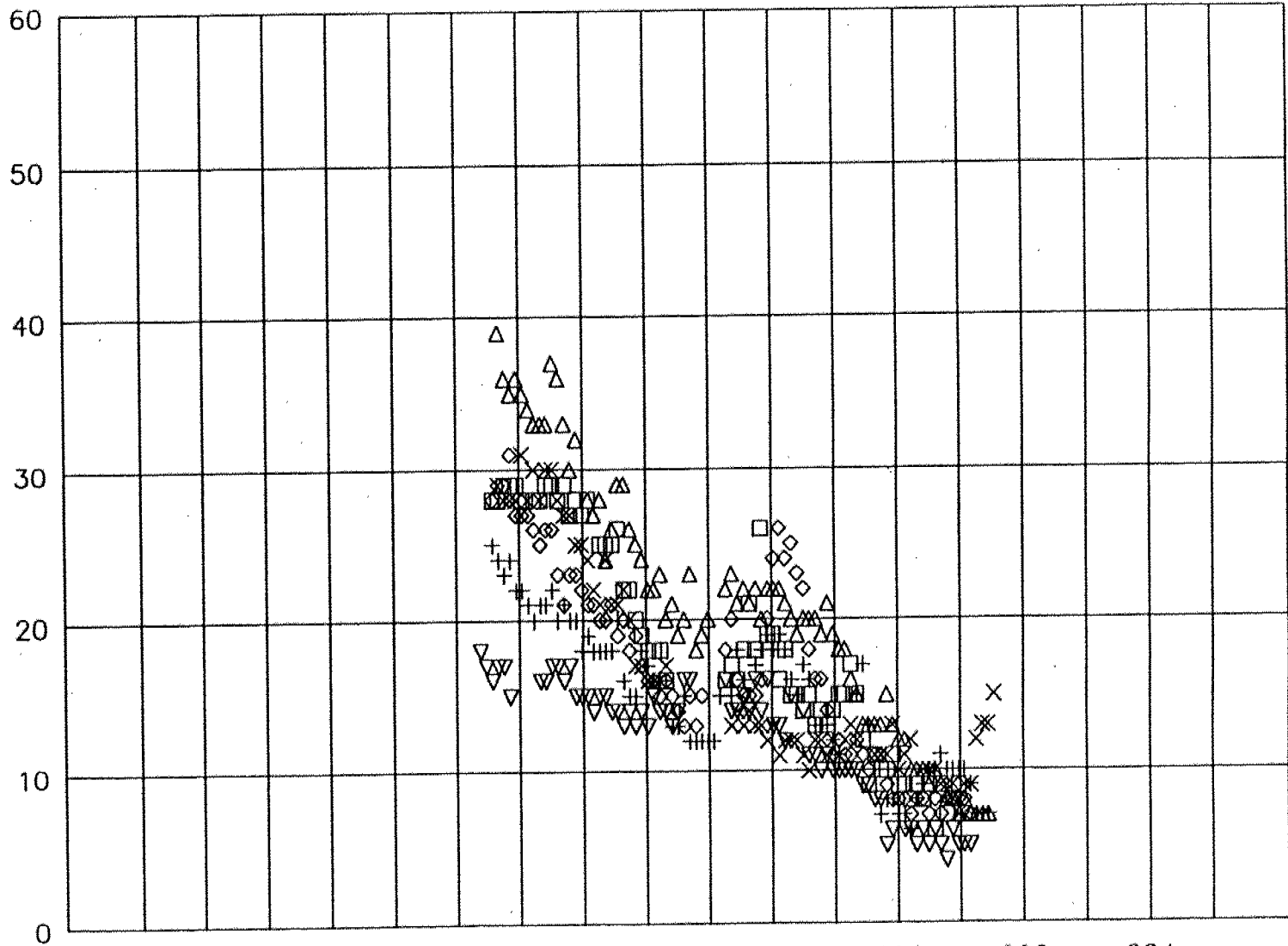


□ PILE B12 + PILE B13 ◇ PILE B14 △ PILE B15 × PILE B16

HWY 400/ LANGSTAFF RD.

EAST ABUTMENT

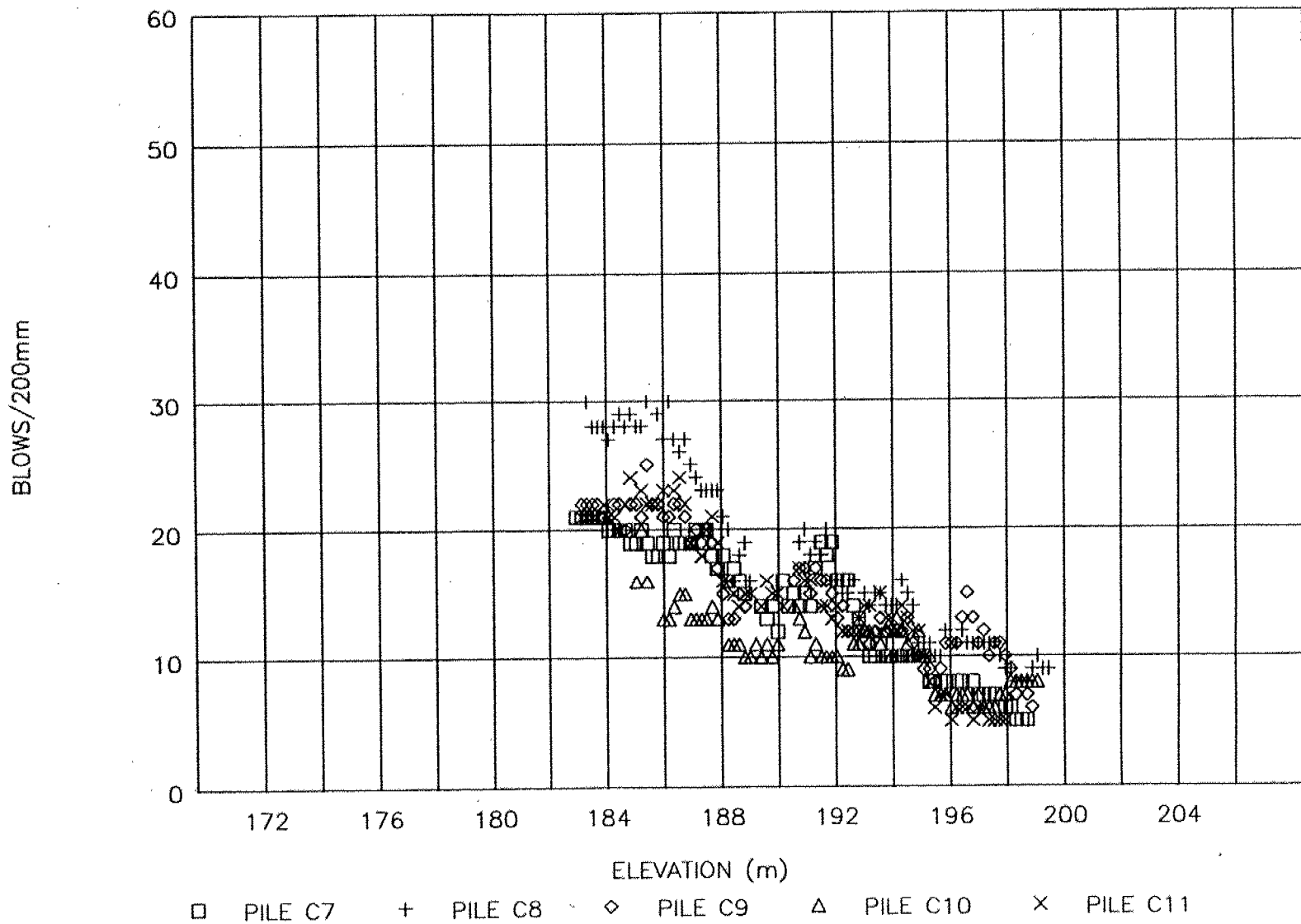
BLOWS/200mm



PILE C1
 PILE C2
 PILE C3
 PILE C4
 PILE C5
 PILE C6

HWY 400/ LANGSTAFF RD.

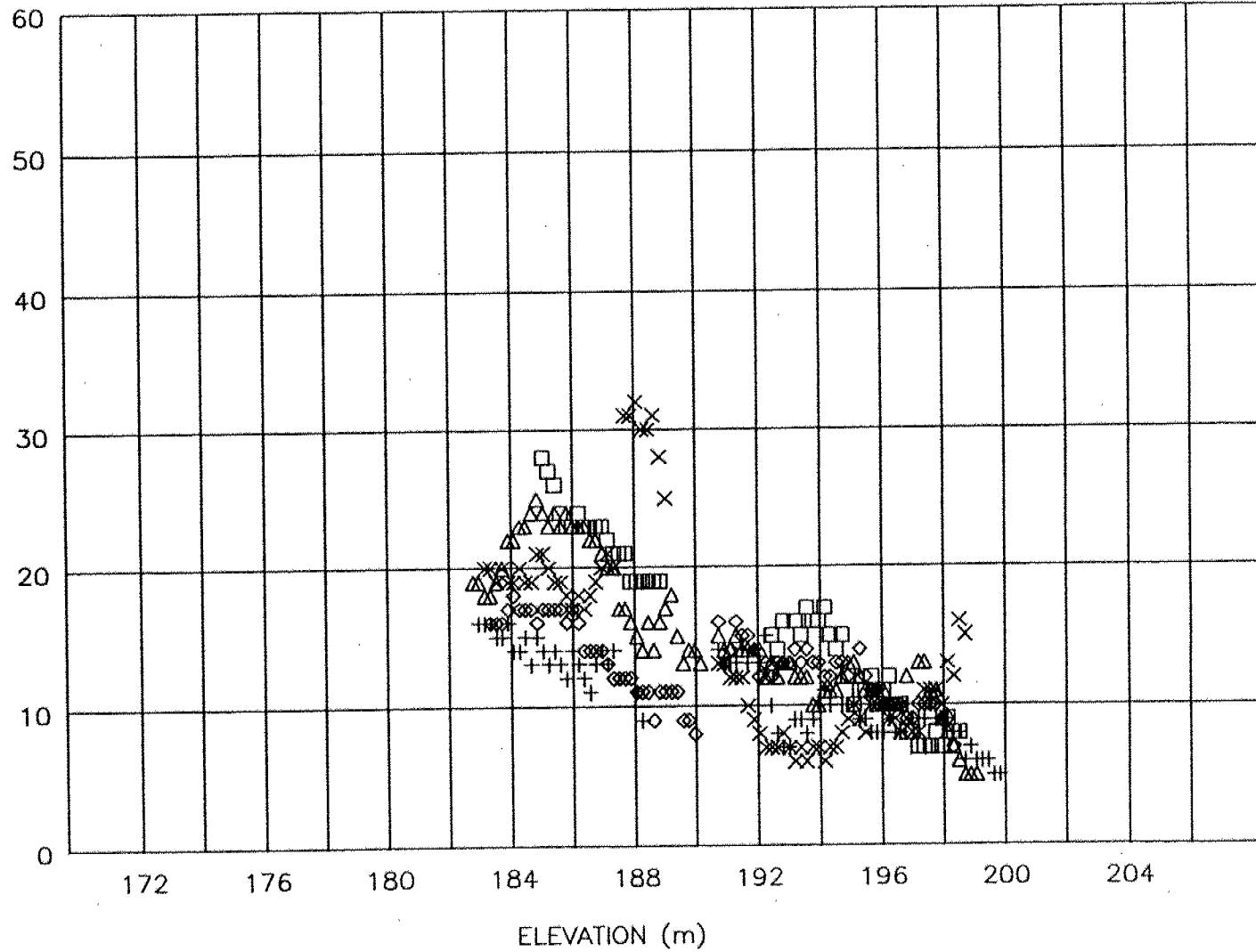
EAST ABUTMENT



HWY 400/ LANGSTAFF RD.

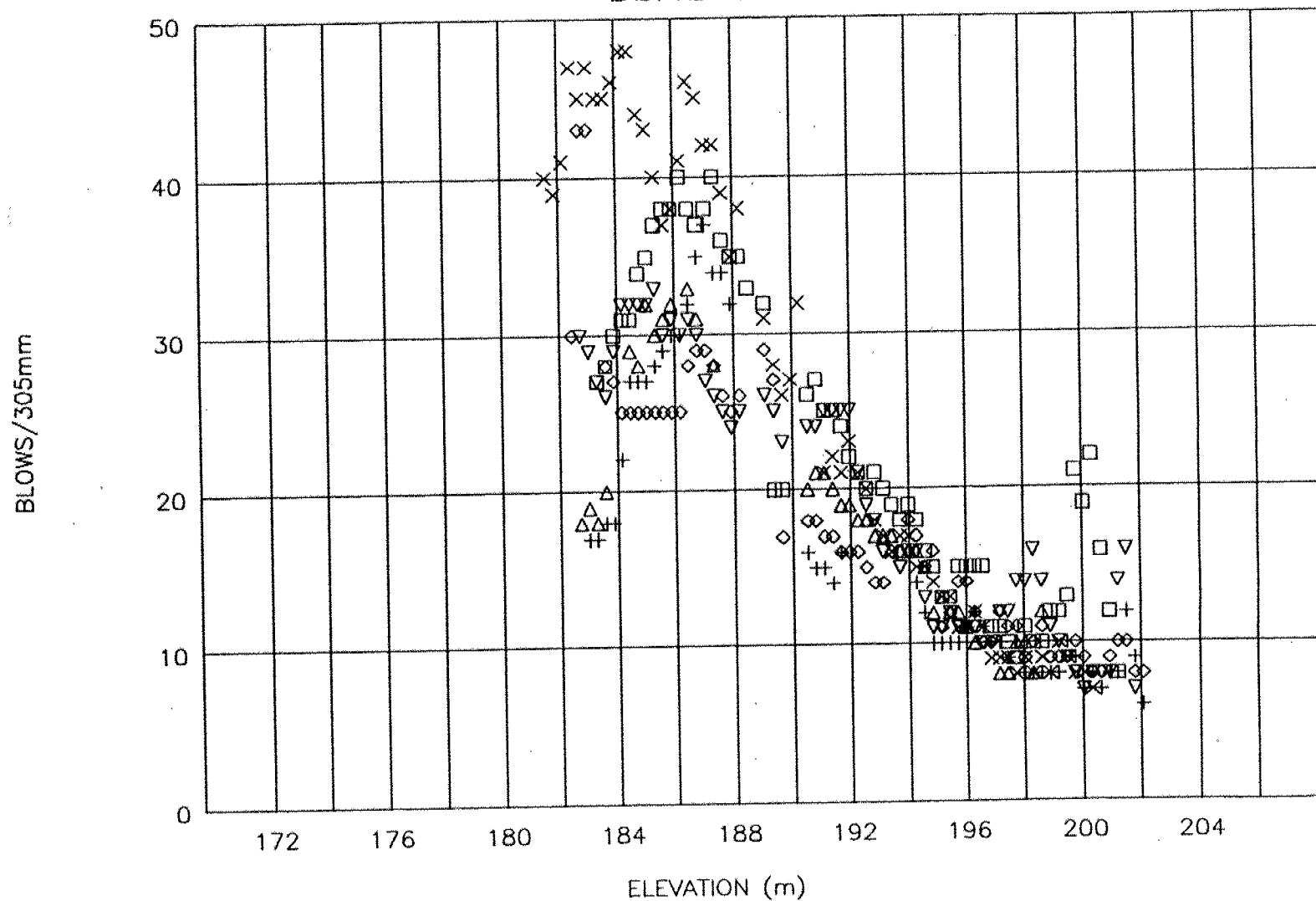
EAST ABUTMENT

BLOWS/200mm



□ PILE C12 + PILE C13 ◇ PILE C14 △ PILE C15 × PILE C16

HWY 400/ LANGSTAFF RD. EAST ABUTMENT

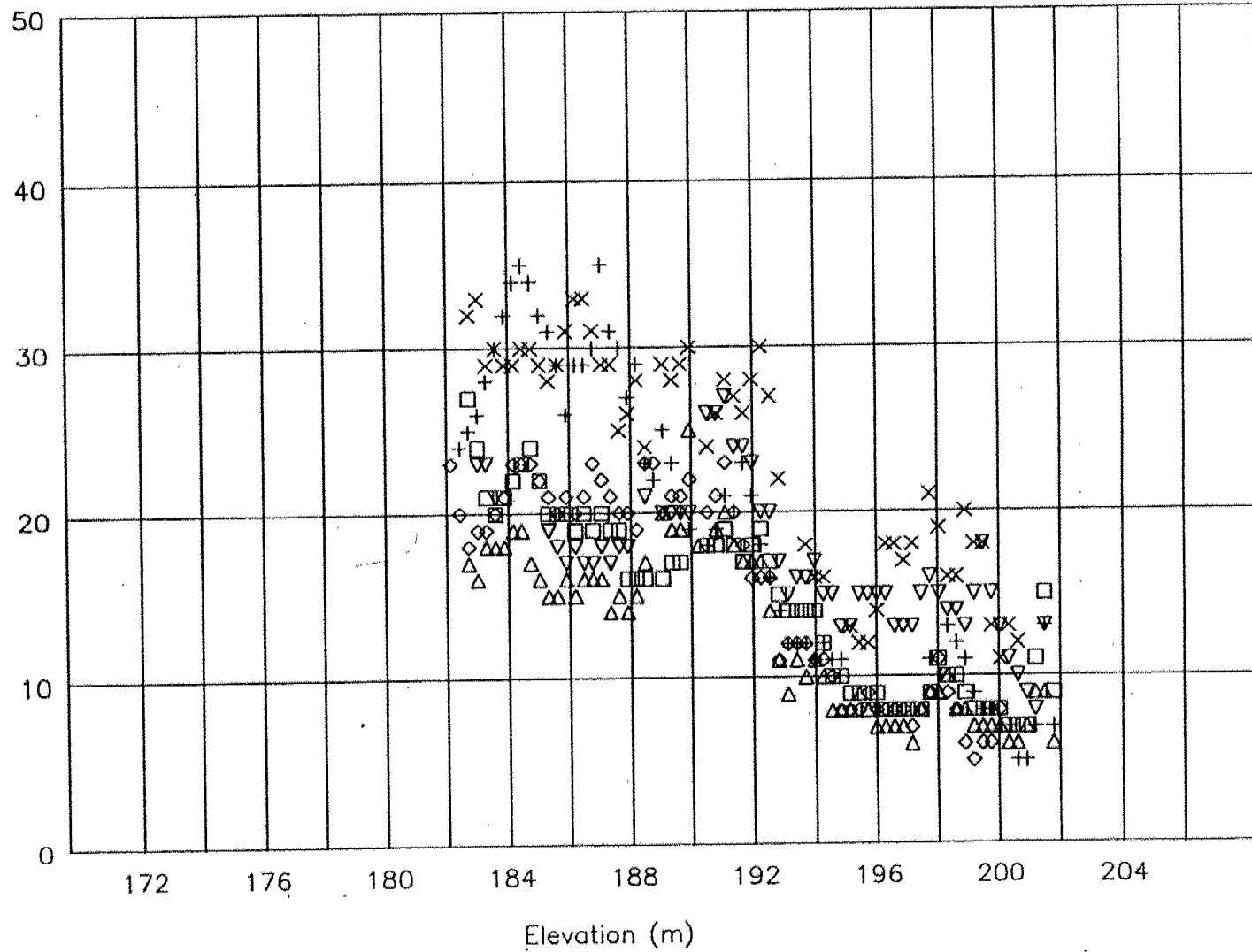


□ PILE D1 + PILE D2 ◇ PILE D3 △ PILE D4 × PILE D5 ▽ PILE D6

HWY 400/ LANFSTAFF RD.

EAST ABUTMENT

Blows/305mm

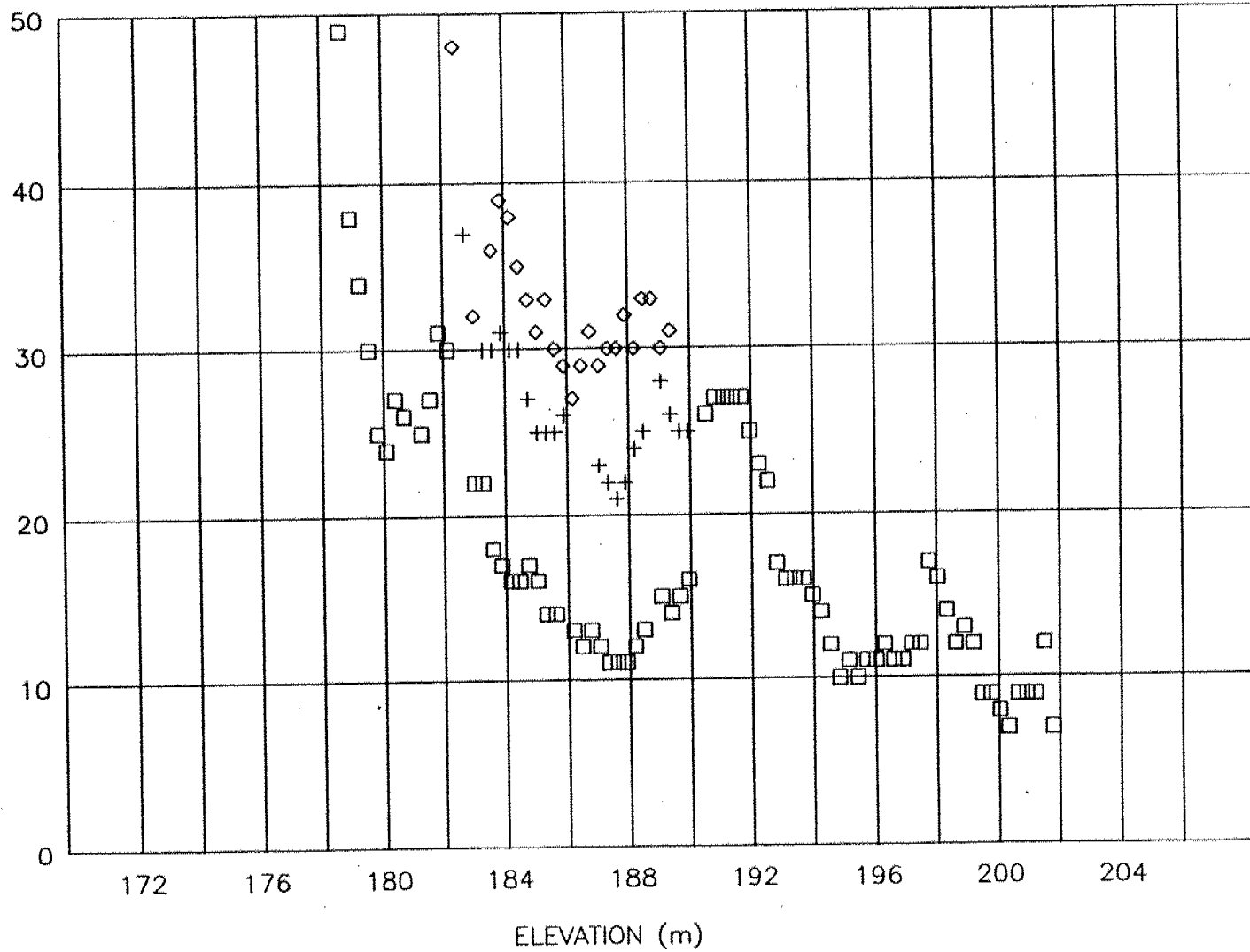


PILE D7
 PILE D8
 PILE D9
 PILE D10
 PILE D11
 PILE D12

HWY 400/ LANGSTAFF RD.

EAST ABUTMENT

BLOWS/305mm



□ PILE D13 + PILE D15 ◇ PILE D16

memorandum



To: T. Zander
Area Construction Engineer
Construction Office
Central Region

Date: 1991 08 15

Attention: B. Benson

From: Foundation Design Section
Room 315, Central Building

Re: Langstaff Road Underpass
W.O. #90 11-001, Site 37-1298
Hwy. 400, District 6, Toronto

As requested, we have reviewed the Golder report dated April 1991, with its appended Geodynamics Inc. report dated December 18/90, regarding dynamic pile testing at the above-noted project. These reports are returned under cover of this memo.

Reference is made to our previous comments


- to Central Region Structural Section in a memo dated 89 03 14,
- and to Central Region Construction Office in a memo dated 90 11 30 (attached).

To summarize:

- We expressed concern during the design stage that a pile foundation may not be the most economical design. If this had been an MTO project it most probably would have been founded on spread footings.
- The use of the pile analyzer to control pile driving is not in accordance with MTO practice at this time, and our experience based on comparisons with static pile load tests has indicated that the pile analyzer is an expensive procedure that presently offers information only marginally superior to the Hiley formula. It would be interesting to know the additional cost due to dynamic monitoring. We estimate it would be in the order of \$40K including both the Golder and Geodynamics input. This amount would be equivalent to installing over 200 m of piles and we would argue that it would be difficult to justify such a large expenditure on pile monitoring at this project.

- Pile driving resistances of 20 blows per foot, as accepted by the consultant, would not be acceptable at MTO regardless of what their pile analyzer predictions indicate. We would normally anticipate resistances of over 10± blows per inch. However, since we would have founded the bridge on spread footings, the concern regarding pile capacities is not critical at this site since the pile caps would probably support the bridge. Also the design capacities of the piles are considerably lower than an MTO design which also provides for an additional safety margin albeit at an additional cost.
- We recommend that future projects funded by MTO should conform to MTO procedures in order to ensure the most cost-effective design.

If there are any questions, please call.


D. Dundas, P. Eng.
Sr. Foundation Engineer
for

M. Devata, P. Eng.
Chief Foundation Engineer

memorandum



To: T. Zander
Toronto Area Construction Engineer
Central Region

Date: 1990 11 30

From: Foundation Design Section
Room 315, Central Building

Re: Langstaff Road Underpass
W.O. 89-11001, Site 37-1298
Hwy. 400, District 6, Toronto

As requested by your office we have reviewed the pile driving operation at this project.

It is our understanding that this is an MTO-subsidized municipal project that will be transferred to MTO upon completion

The Foundation Design Section was originally involved in the project in March/89 when we responded to a request from the Structural Section to review the proposed design. However, after we submitted our comments, there was no further involvement by our office in the design review process. Consequently we were not in a position to ensure consistency with MTO procedures.

In October, 1990 your office requested our assessment of the pile driving operations. At that time the design consultant was providing construction supervision for pile driving based on pile analyzer results. Piles were being driven by an S-70 hydraulic hammer to a depth in the order of 20 m and to a final set of approximately 20 blows per foot.

As part of our review we met with the structural design consultant (McCormick Rankin) and the geotechnical consultant (Golder Assoc.). McCormick Rankin was requested to clarify the design loads for piles at each foundation element while Golder was requested to review their assessment of pile driving requirements based on our concerns as follows:

- the S-70 hydraulic hammer is not yet approved for MTO projects
- the pile analyser is not approved by MTO as an instrument for pile driving control
- pile sets are more typically in the order of 20 blows per inch rather than 20 blows per foot
- the high energy hammer may be damaging the tops of piles

McCormick Rankin responded with their assumed design pile loads, while Golder responded that in their opinion the design pile loads were being achieved.

Based on these responses, we recommend that the piling operation should proceed as directed by McCormick Rankin and Golder. However, we also recommend that piles should be monitored with the Hiley Formula and that these results should be reviewed by our office.

If there are any questions, please call.

A handwritten signature in dark ink, appearing to read "D. Dundas". The signature is written in a cursive, slightly stylized font.

D. Dundas, P. Eng.
Sr. Foundation Engineer

DD/mmj



Golder Associates Ltd.
CONSULTING ENGINEERS

October 19, 1990

Our ref: 901-1404

McCormick Rankin & Associates Limited
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

ATTENTION: Mr. J.L. Malcolm, P.Eng.

FAXED
23/10/90

RE: DYNAMIC MONITORING RESULTS
EAST ABUTMENT PILES
LANGSTAFF ROAD BRIDGE OVER HIGHWAY 400
TOWN OF VAUGHAN, ONTARIO

Dear Sirs:

This letter presents the results of the dynamic monitoring and testing carried out at the above site on October 4, 1990. The testing was carried out by Anna GeoDynamics Inc. on four piles installed in Row D at the east abutment. Pile installation has continued at the east abutment and currently about two-thirds of the 64 piles are in place.

The subsoils encountered in the borehole put down at the location of the east abutment (our report 881-1437) consist of about 17 m of a clayey silt deposit with occasional sandy silt interlayers underlain by about 1.5 m of silt which is in turn underlain by about 6.5 m of a compact to dense sand deposit. The sand deposit extends from about Elevation 187.5 m to 181 m and is underlain by very stiff to hard silty clay. It was recommended in the above report that the piles at the east abutment be founded in the sand deposit.

Pile installation was commenced on October 1, 1990, and is being carried out by Bermingham Construction Ltd. using a hydraulic hammer IHC Hydrohammer S-70 with hammer energy set at 50 kJ to 55 kJ. The dynamic monitoring was carried out on Piles D5, D10, D11 and D13. Details of the driving, tip elevations and monitoring results are given on the attached Table I. In summary, Piles D5, D10 and D11 were driven to similar depths but with variable final sets (17 to 40 blows per 0.3 m) and are terminated within the sand deposit. Testing of Piles D5 and D10 was carried out two days after installation while testing of D11 was completed one day after installation. On the day of the testing, Pile D13 was driven about 5 m deeper than the other test piles and was terminated at a higher initial set (80 blows per 0.3 m) within the silty clay deposit. This pile was tested initially

immediately after driving and then again about 4 hours following completion of driving.

The Pile Driving Analyzer measures force and velocity at the pile head when the pile tip is displaced during striking. Data are recorded during a number of strikes (blows) of the pile and specific blow measurements are then selected for analyses. The dynamic measurements establish the energy actually transferred to the pile, the impact force delivered by the hammer and the maximum force in the pile. The piles were generally tested with the hammer energy set at 40 kJ and the dynamic monitoring indicated that about 85 % to 95 % of the energy was being transmitted to the piles during restrike. During pile installation, therefore, with the energy set at 50 kJ to 55 kJ, the transmitted energy would range from about 43 kJ to 52 kJ.

The energy and force measurements are used to provide an estimation of the ultimate static pile capacity (the Case Method Estimate) of combined shaft and toe resistance. The Case Method Estimates indicate that the ultimate pile capacities for Piles D5, D10 and D11 which are terminated in the sand deposit vary from 1950 kN to 2170 kN. Although the sets at the end of driving of Piles D10 and D11 are substantially different, the load capacities of these piles are similar since the redrive sets during monitoring are about the same. The capacity of Pile D13 terminated in the silty clay deposit is indicated to be 1715 kN immediately following driving and 2010 kN after a four hour set up time.

The Case Method Estimate capacities are generally considered to be accurate to within 10 per cent of the actual load capacity with the variation being dependant mainly on the site subsoil conditions and the ease of driving. A further, and more detailed, calibration of the site parameters is accomplished by a CAPWAP analysis (wave mechanics computations) which is carried out on the input from one blow.

The results of the CAPWAP analyses carried out on Pile D10 indicate an ultimate capacity of 1750 kN which is comprised of 1700 kN in shaft resistance and 50 kN in toe resistance. A value of 1850 kN has been given as the upper bound of the analysis. The CAPWAP analysis carried out on Pile D13 indicates an ultimate capacity of 2200 kN which is comprised of 790 kN in shaft resistance and 1410 kN in toe resistance. The difference in the shaft resistance values between the two piles is a function of the time available for soil set up around the pile after completion of driving. The capacity would likely increase with time by at least 900 kN based on the shaft resistance component of Pile D10.

The final set during initial driving of Pile D10 was 17 blows per 0.3 m. For all subsequent piles to date, except D2 and D4, the final set generally ranged from about 23 to 36 blows per 0.3 m. On redrive during testing of Pile D10, the set increased to about 38 blows per 0.3 m. Similar or greater sets were achieved during redrive of Piles

D11 and D5 and the retap measurements carried out on subsequent Piles A8 and B3 indicate sets of 38 blows per 0.3 m.

Two factors at this site influence the load carrying capacity with respect to the driving criteria and acceptance of the piles. The piles terminated within the sands at the east abutment are deriving their capacity through shaft friction within the upper clayey silt and the sand strata. The actual energy transmitted to the pile by the hydraulic hammer is relatively high (85 % to 95 % energy transfer) compared to that generally achieved by diesel hammers (25 % to 50 % energy transfer). The equivalent set expected for piles under similar conditions but driven with a diesel hammer would be at least twice that measured at this site where the load carrying capacity of the pile is achieved at a lower driving resistance.

Based on the final results of the dynamic monitoring, the pile capacities are considered adequate for the design loading at the east abutment. The lowest capacity (CAPWAP) of 1750 kN was established for Pile D10 which had the lowest set during final driving of the piles tested. Based on the driving records obtained to date, only two other piles - D2 and D4 - had similar sets at similar tip elevations. It is understood that the maximum loading of the piles at the east abutment are as follows:

Row A (western most row):	732 kN	SLS;	951 kN	ULS
Row B :	645 kN	SLS;	810 kN	ULS
Row C :	559 kN	SLS;	685 kN	ULS
Row D (eastern most row):	451 kN	SLS;	557 kN	ULS

Using the lowest pile capacity measured, load factors of 1.8, 2.2, 2.6 and 3.1 are obtained for Rows A through D with corresponding factors of safety ranging from 2.5 to 3.9 for the design serviceability loads. Piles A3 to A14 were driven at least 2 m deeper than the test piles D10 and D11. Shaft friction values given in the CAPWAP analyses indicate an average value of 77 kN/m was obtained over the length of D10. Using this value, it is considered a load factor of at least 2 is applicable to all piles except A1, A2, A15 and A16 to which a load factor of 1.8 would apply. These piles were terminated, however, at sets of at least 24 blows per 0.3 m and although not confirmed would be expected to yield higher capacities. We have considered the pile capacity at Serviceability Limit States by analysing the settlement of the pile group and have calculated that the settlement is less than 25 mm.

The driving criteria as outlined in our letter dated October 5, 1990, and as subsequently discussed verbally should be used for acceptance of all subsequent piles at the east abutment:

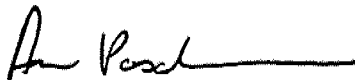
- * It should be ensured that the hammer energy is set at between 50 and 55 kJ during driving.

- * The piles should be driven to at least Elevation 183 m.
- * A minimum resistance of 17 blows per 0.3 m penetration must be achieved over the final 0.3 m of driving.
- * The set should be monitored per 25 mm for a further 100 mm of penetration. If there is a decrease in the number of blows per 25 mm within the initial group of four, the pile should be driven further until there are four consecutive sets/25 mm which are equal to or greater than the previous.
- * At least 20 per cent of the piles, and any pile with final set during driving of less than 20 blows per 0.3 m, should be retapped. If a retap set of less than 13 blows per 100 mm is obtained, the pile must be driven deeper.

We trust that this letter is sufficient for your immediate requirements. Please contact us if you have any questions regarding the recommendations provided.

Yours truly,

GOLDER ASSOCIATES LTD.



A.S. Poschmann, P.Eng.

ASP/dh

Att.: Table 1

TABLE 1

SUMMARY OF DYNAMIC MONITORING RESULTS

PILE	DATE INSTALLED	TIP ELEV. (m)	DRIVING RESISTANCE		ULTIMATE PILE CAPACITY	
			FINAL SET (Blows/25 mm)	REDRIVE SET	CASE METHOD (kN)	CAPWAP (kN)
D5	Oct. 2	181.5	3.3	9	2165	
				9	2170	
				7	2150	
D10	Oct. 2	182.7	1.4	3	1955	1750
				3	2025	
				3	2170	
D11	Oct. 3	182.6	2.7	4	1990	
				4	2015	
				3	1950	
D13	Oct. 4	176.9	6.7	*13	1715	2200
				13	1790	
				*10	2010	

NOTES:

- 1) All piles tested with hammer energy set at 40 kJ except where identified with * indicating 50 kJ setting.
- 2) Final set refers to the driving resistance at the end of initial driving of the pile.
- 3) Redrive set refers to the driving resistance during retapping at the time of the dynamic monitoring.
- 4) For Pile D13, there is a lapse in time of about 4 hours between the first redrive set and the two subsequent sets.

TABLE I PRELIMINARY SUMMARY OF DYNAMIC MEASUREMENTS

PILE and BLOW #	EMBD (m)	EMAX (KJ)	ENERGY RATIO (%)	FIMP (KN)	SIMP (MPa)	FMAX (KN)	SMAX (MPa)	CMES RA2 (KN)	PRES (blows/ 25 mm)	ENGY (KJ)
PILE D5 (LGTH = 24.37 m)										
2 BOR	23.1	29.2	75	3110	220	3110	220	2165	9	40
6 RSTR	23.1	34.5	85	3220	230	3280	230	2170	9	40
23 EOR	23.1	37.9	95	3240	230	3400	240	2150	7	40
PILE D10 (LGTH = 23.41 m)										
1 BOR	21.9	33.5	85	2885	205	2985	210	1955	3	40
6 RSTR	21.9	37.8	95	3310	235	3375	240	2025	3	40
22 EOR	21.9	37.2	95	3115	220	3491	245	2170	3	40
PILE D11 (LGTH = 23.30 m)										
1 BOR	21.9	30.8	75	2935	210	3120	220	1990	4	40
5 RSTR	21.9	38.4	95	3205	225	3550	250	2015	4	40
19 EOR	21.9	39.1	95	3405	240	3475	245	1950	3	40
PILE D13 (LGTH = 29.38 m)										
31 EOID	28.2	45.4	90	3525	250	3525	250	1715	13	50
7 RSTR	28.2	38.0	95	3215	230	3485	245	1790	13	40
29 EOR	28.2	44.0	90	3550	250	3700	260	2010	10	50

LGTH: Length of pile below dynamic monitoring gages
 EMBD: Embedment depth
 EMAX: Maximum transferred energy
 FIMP: Impact force
 SIMP: Impact stress
 FMAX: Maximum compression force
 SMAX: Maximum compression stress
 RA2: Case Method Estimate for pile with shaft and toe resistance
 PRES: Penetration resistance in blows per 25 mm
 ENGY: Nominal energy setting of IHC S70 hammer
 BOR: Beginning-of-restrike
 RSTR: Restrike
 EOR: End of restrike

Final CAPWAPC Capacity: Ru 1750.7, Skin 1698.7, Toe 52.0 kN

Soil Sgmnt No.	Depth Below Gages m	Depth Below Grade m	Ru kN	Sum of Ru Up kN	Sum of Ru Down kN	Unit Resist. w. Respect to Depth kN/m	Resist. Area kN/m2	Smith s/m	Quake mm
				1750.7					
1	3.1	1.5	155.1	1595.6	155.1	76.19	.00	.272	3.000
2	5.1	3.6	130.9	1464.8	286.0	64.29	.00	.272	3.000
3	7.1	5.6	94.7	1370.1	380.7	46.53	.00	.272	3.000
4	9.2	7.7	88.6	1281.5	469.3	43.52	.00	.272	3.000
5	11.2	9.7	118.8	1162.7	588.0	58.35	.00	.272	3.000
6	13.2	11.7	141.2	1021.5	729.2	69.36	.00	.272	3.000
7	15.3	13.8	145.4	876.1	874.6	71.42	.00	.272	3.000
8	17.3	15.8	170.1	706.1	1044.7	83.54	.00	.272	3.000
9	19.3	17.8	215.8	490.3	1260.4	105.99	.00	.272	3.000
10	21.4	19.9	229.7	260.6	1490.2	112.84	.00	.272	3.000
11	23.4	21.9	208.6	52.0	1698.7	102.45	.00	.272	3.000
Average Skin Values			154.4			77.57	.00	.272	3.000
Toe			52.0						
						3689.48	1.129		4.000

Soil Model Parameters/Extensions

		Skin	Toe
Case Damping		.800	.102
Reloading Level	(% of Ru)	0	0
Unloading Level	(% of Ru)	10	
Soil Plug Weight	(kN)		.75

Final CAPWAPC Capacity: Ru 2201.9, Skin 791.6, Toe 1410.4 kN

Soil Sgmnt No.	Depth Below Gages m	Depth Below Grade m	Ru kN	Sum of Ru Up kN	Sum of Ru Down kN	Unit Resist. w. Respect to Depth kN/m	Resist. Area kN/m2	Smith s/m	Quake mm
				2201.9					
1	3.0	1.9	118.5	2083.5	118.5	58.47	.00	.365	6.500
2	5.1	3.9	85.5	1998.0	204.0	42.20	.00	.365	6.500
3	7.1	5.9	47.3	1950.7	251.3	23.34	.00	.365	6.500
4	9.1	7.9	38.1	1912.6	289.3	18.79	.00	.365	6.500
5	11.1	10.0	53.8	1858.8	343.2	26.57	.00	.365	6.500
6	13.2	12.0	57.3	1801.5	400.5	28.29	.00	.365	6.500
7	15.2	14.0	36.8	1764.6	437.3	18.17	.00	.365	6.500
8	17.2	16.0	21.0	1743.6	458.3	10.37	.00	.365	6.500
9	19.2	18.1	44.5	1699.1	502.8	21.97	.00	.365	6.500
10	21.3	20.1	90.3	1608.8	593.1	44.56	.00	.365	6.500
11	23.3	22.1	89.4	1519.4	682.5	44.12	.00	.365	6.500
12	25.3	24.1	39.2	1480.2	721.7	19.36	.00	.365	6.500
13	27.4	26.2	21.2	1459.0	743.0	10.49	.00	.365	6.500
14	29.4	28.2	48.6	1410.4	791.6	23.98	.00	.365	6.500
Average Skin Values			56.5			28.07	.00	.365	6.500
Toe			1410.4			*****		.143	10.000

Soil Model Parameters/Extensions

		Skin	Toe
se Damping		.500	.350
Reloading Level	(% of Ru)	0	0
Unloading Level	(% of Ru)	2	
Resistance Gap	(mm)		1.00
Soil Plug Weight	(kN)		1.25

McCORMICK RANKIN

CONSULTING ENGINEERS

FAX. NO. (416) 823-8503

FACSIMILE TRANSMISSIONTO: Foundation Design Section
Ministry of TransportationATTENTION: Mr. D.H. Dundas, P. Eng.ADDRESS: #315, Central Building1201 Wilson AvenueDownsview, Ontario M3M 1J8FAX NO. 235-5240DATE: Nov. 2, 1990No. of sheets being transmitted: 2 Including this sheet.W. O. No. 1931-600Langstaff Road Underpass at Highway
400. District No. 6
MTO Site No. 37-1298SENT FROM: Mississauga**MESSAGE:**

As requested on October 31, 1990, we have summarized the piling design for the above-noted structure as follows:

a) EAST ABUTMENT

This is designed primarily as an Abutment which can later be used as a Pier. The abutment design loads will govern. The basic abutment cross-section is sized to provide enough dead load (mainly from the backfill on the heel) to counteract the overturning moment produced by the horizontal load from the backfill (10.5 m to the u/s of footing).

Because of the configuration of the abutment the P/A + Mc/I will always result in the front (toe) piles having the highest loading (732 kN SLS) and decreasing to a lower loading at the back of the heel (451 kN SLS). The effect of the direct vertical loading (P/A) on each pile is at 600 kN SLS. The SLS condition governs. The allowable pile loading at SLS is indicated at 750 kN.

b) EAST PIER

Maximum pile loading 1323 kN (ULS) and 833 kN (SLS). The piling layout is governed by ULS condition of 1323 kN. Reduction of the layout by one set (2 piles) would raise the applied pile loading over the indicated allowable limit.

c) WEST PIER

Maximum pile loading 1070 kN (ULS) and 910 kN (SLS). The piling layout is governed by the SLS condition and a revision to the layout would raise the applied pile loading over the allowable SLS limit.

d) WEST ABUTMENT

Maximum pile loading 955 kN - ULS, 752 kN - SLS. The piling layout was set up similar to the east abutment. The height of the west abutment to u/s footing is 6.5 m. The SLS condition governs.

We would appreciate your review of the enclosed information. If there are any questions or comments, please call us.

Yours very truly,

McCORMICK RANKIN



K. Woon-Fat, P. Eng.

**Golder Associates Ltd.**
CONSULTING ENGINEERS

OCT 10 1990

October 5, 1990

Our ref: 901-1404

McCormick Rankin & Associates Limited
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

ATTENTION: Mr. J.L. Malcolm, P.Eng.

RE: PRELIMINARY RESULTS OF DYNAMIC MONITORING
EAST ABUTMENT PILES
LANGSTAFF ROAD BRIDGE OVER HIGHWAY 400
TOWN OF VAUGHAN, ONTARIO

Dear Sirs:

This letter summarizes the preliminary results of the dynamic monitoring and testing carried out at the above site on October 4, 1990. The testing was carried out by Anna GeoDynamics, a specialist testing company, on four piles installed in Row D at the east abutment. Our report providing details of the testing together with an engineering interpretation of the testing will be submitted on receipt of the final test results from Anna GeoDynamics Inc.

The preliminary summary of dynamic measurements received indicate pile capacities at 'end-of-restrike' ranging from 1950 kN to 2170 kN with the IHC Hrdrohammer S-70 hydraulic hammer delivering a measured energy of 37 kJ to 39 kJ. This range in capacity is considered adequate for the piles at the east abutment where it is understood the maximum loading at Ultimate Limit States is 950 kN.

It is understood that the piles were initially driven with the hammer energy set at 50 kJ to 55 kJ. The piles were tested with the hammer energy set at 40 kJ and the monitoring indicated that generally about 85 % to 95 % of the energy was being transmitted to the piles during restrike. Based on these results and the driving records for the piles tested, we recommend that the following driving criteria be used for acceptance of all subsequent piles at the east abutment:

- * It should be ensured that the hammer energy is set at between 50 and 55 kJ.
- * The piles should be driven to at least Elevation 183 m.

October, 1990

2

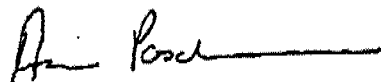
901-1404

- * A minimum resistance of 20 blows per 0.3 m penetration must be achieved over the final 0.3 m of driving.
- * The set should be monitored per 25 mm for a further 100 mm of penetration. If there is a decrease in the number of blows per 25 mm within the initial group of four, the pile should be driven further until there are four consecutive sets/25 mm which are equal to or greater than the previous.

We trust that this letter is sufficient for your immediate requirements. Please contact us if you have any questions regarding the recommendations provided.

Yours truly,

GOLDER ASSOCIATES LTD.



A.S. Poschmann, P.Eng.

ASP/dh

McCORMICK RANKIN

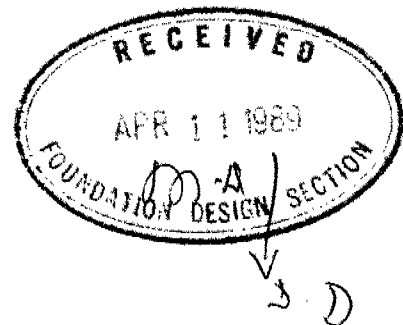
CONSULTING ENGINEERS

April 6, 1989

Mr. G.C.E. Burkhardt, P. Eng.
Head, Structural Section
Central Region
Ministry of Transportation Ontario
5000 Yonge Street
Willowdale, Ontario
M2N 6E9

Attention: Mr. D.N. Bye
Structural Supervisor

RE: Town of Vaughan
Langstaff Road Underpass
Site 37-1298
Hwy. 400, District 6
Our File: W.O. 1931-88



Dear Sir:

We are in receipt of your letter, dated 1989-03-21, summarizing the comments arising from the Ministry's review of the Preliminary General Arrangement Drawing for the above-noted structure.

The main points arising from this review are associated with the foundation design for the project. We have discussed the points raised by the Ministry's Foundation Design Group with our geotechnical sub-consultants, Golder Associates. The attached letter report, dated March 28, 1989, summarizes Golder Associates' consideration of the points raised and provides additional parameters and recommendations where requested.

However, the main issue is whether or not the use of steel H-piles to support the piers and abutments is the most appropriate and cost-effective solution for the foundations at this site. On the basis of Golder Associates' investigation and the difficulties associated with spread foundations outlined in the attached submission, we are of the opinion that the use of steel 'H' piles at all piers and abutments is the correct decision for this project. Accordingly, the design has now been completed on this basis.

McCORMICK RANKIN

Mr. G.C.E. Burkhardt, P. Eng.
April 6, 1989

Page 2

If there are any further questions or concerns regarding any aspect of this submission, please do not hesitate to telephone.

Yours very truly,

McCORMICK RANKIN

RS:dw

Encl.

cc: Mr. W. Robinson, P. Eng.
Mr. M. Devata, P. Eng.
Mr. M. Holowka, P. eng.

Dr. R. Skelton, P. eng.



Golder Associates Ltd.

CONSULTING ENGINEERS

MAR 30 1989

March 28, 1989

Our ref: 881-1437

McCormick, Rankin & Associates Limited
Consulting Engineers
60 Briarwood Avenue
MISSISSAUGA, Ontario
L5G 3N6

ATTENTION: Mr. R. Skelton

RE: LANGSTAFF ROAD UNDERPASS
SITE 37-1298
HWY 400, DISTRICT 86

Dear Sirs:

Further to your request, we have reviewed the comments on the above project provided by the Ministry of Transportation of Ontario, Structural Section, in their letter dated March 21, 1989. Reference is made to our report 881-1437, dated February 1989.

Shallow Foundation Considerations

It has been suggested that consideration be given to supporting the abutment footings on bank seats within the approach embankments fills and supporting the piers on shallow spread footings.

Bank seat construction involves placement of Granular A fill material compacted to 100 per cent Standard Proctor within a zone as defined in the attached Figure 1. The factored bearing capacity at ULS for footings constructed as indicated in the figure may be taken as 800 kPa. The coefficient of friction between the footing and the granular fill may be taken as 0.46 and 0.55 at ULS and SLS, respectively. The settlement of footings constructed within the embankment is relatively insensitive to the bearing pressure applied but is directly proportional to the settlement of the embankment fill and of the subsoils below the embankment. It is expected that the settlement of compacted Granular A will be completed by the time the footings are constructed however consolidation of the silty clay foundation soils will be ongoing under the embankment loading. It is estimated that up to 80 mm of settlement of the footing due to consolidation of the 3 m thick deposit of silty clay could occur.

If the bridge design cannot withstand this magnitude of settlement, subexcavation of the silty clay deposit (up to 3 m thick in the vicinity of the abutments) and replacement with compacted, clean free draining fill is required.

Based on the results of Boreholes 3 and 4 put down at the pier locations, it is considered that the highest suitable founding levels are Elevation 201.3 m (4 m depth) at the west pier and Elevation 202.3 m (4.7 m depth) at the east pier. This would result in a requirement for excavations as much as 5 m deep within the highway median for spread footing construction at the east pier location. The founding soils, as encountered in the boreholes, would consist of a sandy silt till deposit which, at these elevations, is under a piezometric pressure of about 2.7 m head of water. The silt till is susceptible to softening and/or disturbance when subjected to uplift pressures and exposed to water. As part of our investigation, the grain size distribution was determined on only one sample of the silt till obtained from Borehole 3. The gradation indicates 55 per cent passing the #200 sieve and about 15 per cent clay sizes. Although the percentage of clay sizes is probably adequate overall to inhibit excessive flow of groundwater into the excavation, larger quantities would be expected where more permeable layers are encountered within the till.

The silt till at the location of Borehole 4 is underlain at 5.5 m depth (1.5 m below the proposed founding level) by a layer of silt at least 1 m thick. This material is considered to be extremely susceptible to disturbance by water seepage and, with the inherent variability of subsurface conditions at this site, further subsurface investigation would be required to confirm if this silt layer is only local in extent. If the silt is found to be a significant layer at the proposed founding elevation, dewatering prior to excavation would be required to reduce the water pressures at the base of the excavation. Dewatering of this layer would be difficult; eductors placed within the silt layer would have the best chance of success. For these reasons, we do not recommend the use of spread foundations where the founding level is close to an extensive layer of silt.

If the silt layer can be shown by further investigation, not to seriously affect the performance of spread footings, a factored capacity at ULS of 450 kPa may be assumed for spread footings founded on undisturbed silt till at the above noted elevations. The bearing capacity at SLS may be taken as 300 kPa. The coefficient of friction between the silt till and the concrete may be assumed to be 0.28 and 0.35 at ULS and SLS, respectively.

Retaining Walls

It is understood that retaining walls will be constructed when the future bridge extension to the east is carried out. We have not been requested to address this issue for this phase of the investigation - reinforced earth retaining structures are likely feasible with some subgrade preparatory work.

Granular Backfill

Due to the variability in Granular B from various sources, a wide range of values for unit weight and effective angle of internal friction are possible. The failure mode of the fill materials behind the abutment is also influenced at this site by the nature of the embankment fill materials located outside the zone of granular fill. For design, a unit weight equal to 20.5 kN/cu.m may be assumed for the backfill to the abutment. The equivalent earth pressures given in our report can be used; these assume factors of 0.58 and 0.48 for ULS and SLS, respectively, in the 'at rest' condition and 0.4 and 0.32 for ULS and SLS in the 'active' condition.

Embankment Stability and Settlement

For embankments constructed of clean earth fill compacted to 95 per cent of the materials Standard Proctor dry density, side slopes formed at 2 horizontal to 1 vertical are considered to be appropriate provided that they are topsoiled and seeded.

Settlement of the approach embankments due to consolidation under self weight will be dependent on the type of fill materials used. For typical well compacted silt till materials of low plasticity, about 80 mm of settlement would be anticipated for 8.5 m high embankments. Additional settlement due to consolidation of the underlying silty clay deposit is estimated at 50 mm for a 2 m thickness of clay as encountered at the locations of Boreholes 2 and 5.

Excavations and Temporary Shoring

Excavations for pile cap construction will generally be carried through silty clay to clayey silt deposits and will probably extended to just below the groundwater level. Excavations for footing construction if adopted for the piers will be carried through fill, silty clay and sandy silt till deposits and will be about 2.7 m below

the groundwater level. Where space permits, excavations through these materials may be carried out with side slopes formed no steeper than 1.5 horizontal to 1 vertical. Excavations for the piers, in the highway median and adjacent to the highway, should be carried out within soldier pile and lagging wall systems, braced by prestressed struts.

Groundwater flow into the excavations through the silty clay/clayey silt deposits is expected to be minimal and can be handled by pumping from properly filtered sumps within the excavation. The silt till founding stratum, however, is susceptible to softening and provision for ensuring adequate protection of the excavation base is required. A working mat consisting of a layer of compacted well graded free draining granular fill, such as Granular A fill should be placed over the prepared excavation base after inspection of the base is complete.

Soldier piles should be concreted into pre-augered holes. Struts should be prestressed to 120 per cent of their design load. The upper struts should be installed no deeper than 1 m below ground surface and they should be stressed before any further excavation is carried out. Lagging boards should be installed as soon as space permits and the space behind the lagging should be packed with lean concrete.

The temporary support system should be designed to resist the horizontal soil loading, hydrostatic loading and surcharge from highway traffic as shown on the attached Figure 2. The unit weight of the retained soil may be taken as 19 kN/cu.m. A coefficient of lateral earth pressure equal to 0.4 should be assumed. The water level should be taken at Elevation 204 m. Temporary toe support to the pile sockets within the dense/very stiff silt till deposits below the water table may be calculated from the expression:

$$R_p = 7.5 (B) (D^2) (K_p)$$

where

R_p = allowable passive toe resistance (kN)
 B = socket diameter (m)
 D = socket depth (m)
 K_p = coefficient of passive pressure
= 4.5 for socket in till

The above expression applies a factor of safety of 2 to the passive pressure.

We trust that the information contained in this letter adequately addresses the comments provided in the Ministry's letter of review. If any of the information requires further clarification, please contact us.

Yours truly,

GOLDER ASSOCIATES LTD.



A.S. Poschmann, P.Eng.



J.R. Busbridge, P.Eng.

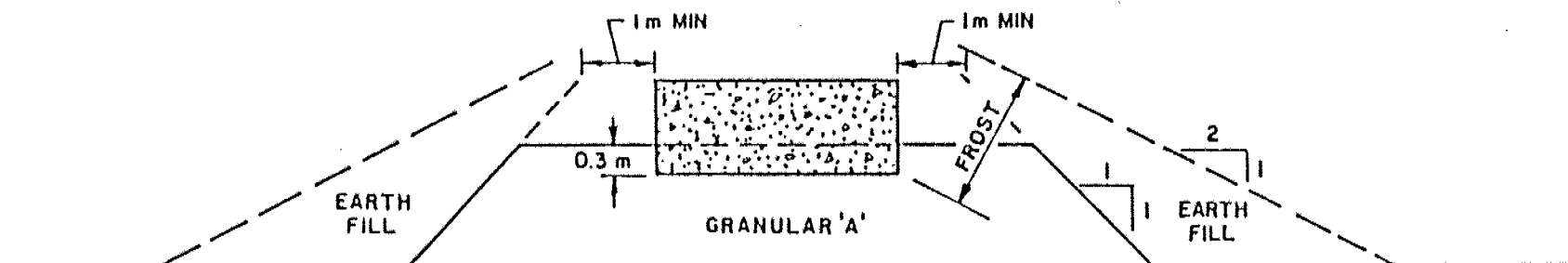
ASP/JRB/ga

Att.: as noted above

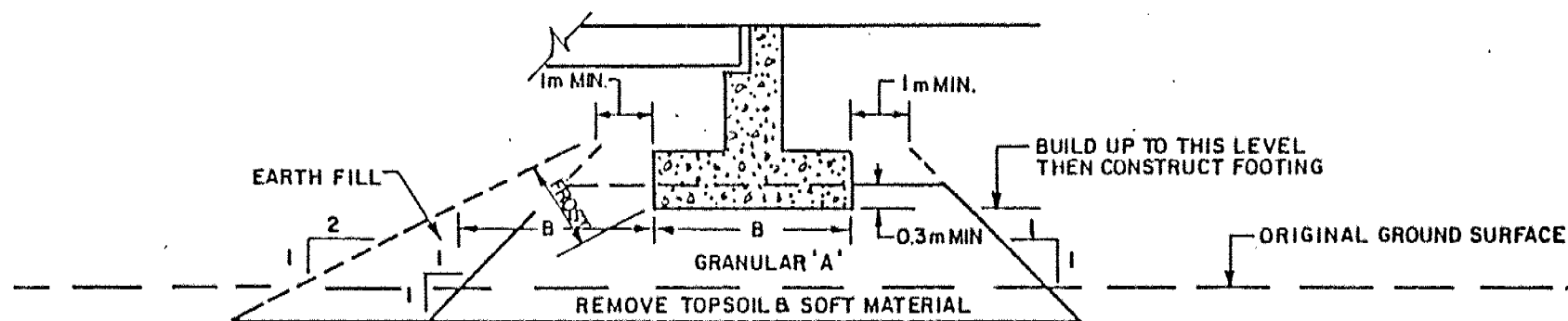
Date: MARCH 1989
Project: 881-1437

Golder Associates

Drawn: TDR
Chkd: AY



CROSS - SECTION



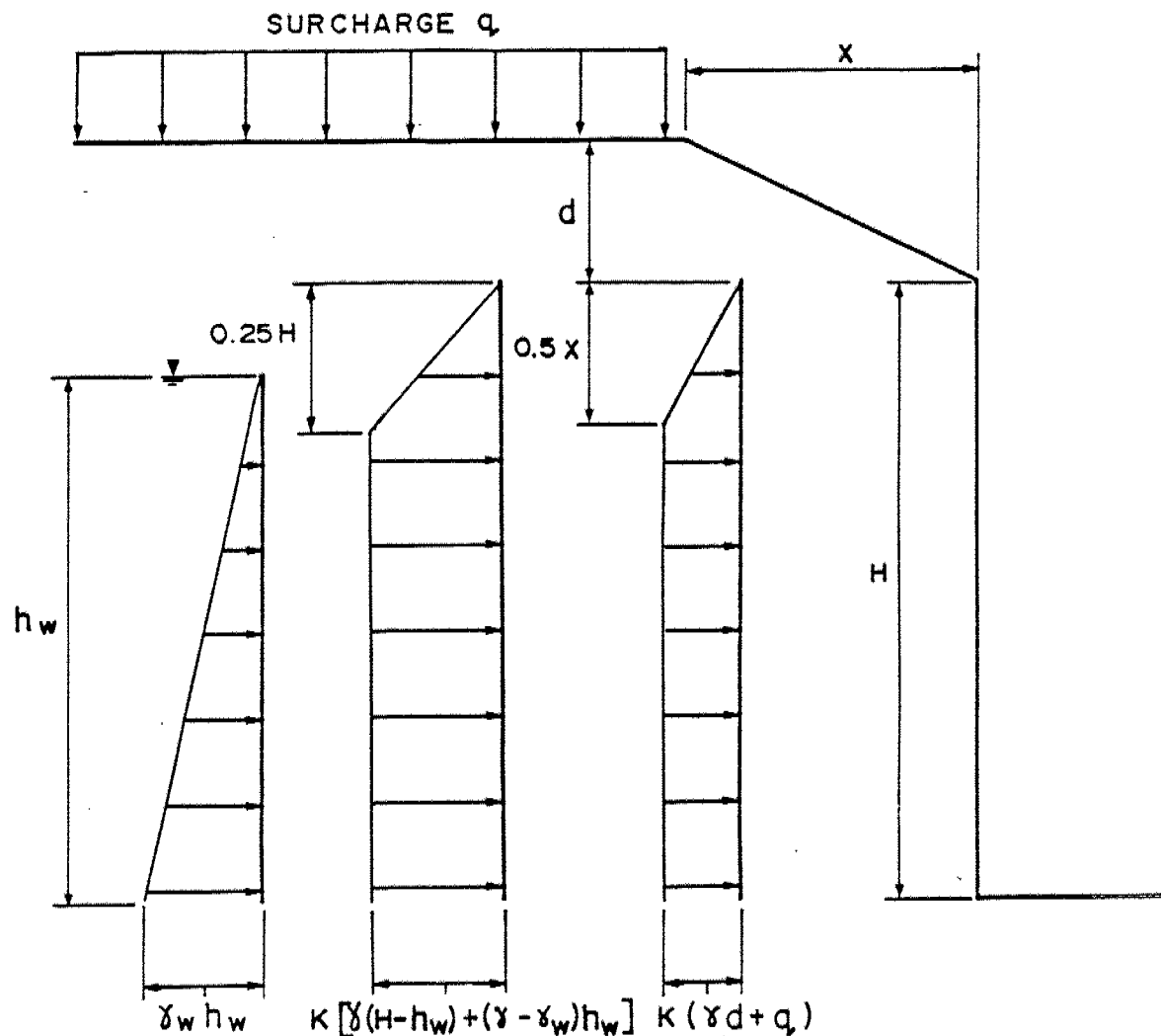
LONGITUDINAL SECTION

NOT TO SCALE

- NOTES:
1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
 2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.C. STANDARDS.
 3. CONSTRUCT CONCRETE FOOTING
 4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED
 5. SOURCE M.T.C. 1982

ABUTMENT COMPACTED FILL SHOULDER
GRANULAR 'A' CORE

FIGURE 1



γ = UNIT WEIGHT OF SOIL

γ_w = UNIT WEIGHT OF WATER

K = EARTH PRESSURE COEFFICIENT

(REFER TO TEXT OF REPORT FOR DESIGN VALUES)

N.B. WHERE THERE IS NO SLOPING GROUND BEHIND WALL, $d = 0$

Date MARCH 1989
Project 881-1437

Golder Associates

Drawn TDR
Chkd. P&P



Ontario

Ministry
of
Transportation

Ministère
des
Transports

Structural Section
Central Region
5000 Yonge Street
Willowdale, Ontario
M2N 6E9

Telephone: 224-7426

1989-03-21

Mr. R. McCormick
McCormick, Rankin
Consulting Engineers
60 Briarwood Avenue
Mississauga, Ontario
L5G 2N6

Attention: R. Skelton

Re: Langstaff Road Underpass
Site 37-1298
Hwy 400, District 6

The General Arrangement Drawing for the above mentioned structure has been reviewed by the Region, Structural head Office and the Foundation Design Section. Comments received are indicated on the enclosed plan or as listed below. The Foundation Design Section also reviewed the Foundation Investigation Report.

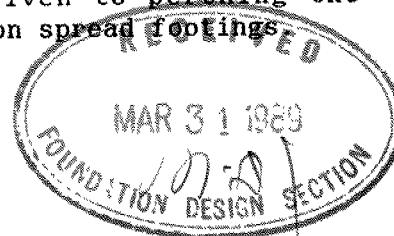
M. Holowka, Head Approvals Section, Structural Office

1. Use at-rest pressures for the abutment design if the piles are less than 5 metres in length.
2. Use a shear block at the abutments to resist transverse forces.

M. Devata, Chief Foundation Engineer, Foundation Design Section

The following comments from the Foundation Design Section are offered as suggestions for your consideration.

1. The present proposal has piers and abutments supported on steel H-piles. Although this may be a feasible solution, we are concerned that end-bearing strata have not been adequately defined, and that the consultants requirements for their on-site supervisor of dynamic monitoring or a minimum of 2 full-scale static load tests may not be cost-effective. In our opinion, consideration should instead be given to perching the abutments on granular pads and founding the piers on spread footings.



If spread footings are considered, founding elevations and bearing capacities are available for your use. Recommendations for sliding resistance and working slabs would be required.

2. If retaining walls are required, consideration should be given to reinforced earth retaining structures.
3. M.T.O. practice is to specify material parameters for granular backfill rather than provide equivalent fluid pressures.
4. Although deep-seated instability of the 8.5 m approach fills is not anticipated, the surficial stability of the high fill should be addressed.
5. Since groundwater elevations have been reported to be higher than proposed excavations and the non-cohesive strata are susceptible to disturbance, dewatering recommendations are required.
6. Recommendations for the proposed roadway protection scheme are required.
7. Recommendations for anticipated settlement under the approaches should be provided.

If the above comments require further input from the Foundation Design Section, they have indicated that they would be available at a mutually agreeable time.

If trust this data will be of assistance to you in completing the design.

D.H. Bye
Structural Supervisor
for:
G.C.E. Burkhardt
Head, Structural Section

DHB:dd

cc: M. Holowka

S. Jacobs

M. Devata ✓

memorandum



To: Mr. G.C.E. Burkhardt
Head, Structural Section
Central Region

Date: 1989 03 14

Atten: D.H. Bye
Structural Supervisor

From: Foundation Design Section
Room 315, Central Building

RE: Langstaff Road Underpass
W.O. 89-11001, Site 37-1298
Hwy. 400, District #6, Toronto

As requested in your memo dated February 24, 1989, we have reviewed the preliminary General Arrangement Drawing and Foundation Report for this project. Our comments follow. However, please note that this memo is for internal M.T.O. use only. If you require our input in negotiations with the municipality or its consultants, we could be available to participate at a mutually convenient time.

It is our understanding that a 3-span structure with 8.5 m high approaches has been proposed to carry Langstaff Road over Hwy. 400. The existing ground surface is flat at elev. 205.0 \pm m while the Hwy. 400 embankment is at elev. 206.5 to 207.0 m. The proposal has established tops of pier footings at elev. 205.0 m.

The present proposal has piers and abutments supported on steel H-piles. Although this may be a feasible solution, we are concerned that end-bearing strata have not been adequately defined, and that the consultants requirements for their on-site supervisor of dynamic monitoring or a minimum of 2 full-scale static load tests may not be cost-effective. In our opinion, consideration should instead be given to perching the abutments on granular pads and founding the piers on spread footings. At the abutments, we would anticipate that the granular pads would be founded at elev. 202.5 and that bearing capacities would be in the order of

- Factored Bearing Capacity at U.L.S. = 900 kPa
- Bearing Capacity at S.L.S. Type II = 350 kPa

At the piers, we would anticipate that the footings would be founded on native overburden at elev. 202.5 and elev. 201.0 at the east and west piers respectively, with bearing capacities in the order of

- Factored Bearing Capacity at U.L.S. = 525 kPa
- Bearing Capacity at S.L.S. Type II = 350 kPa

If spread footings are adopted, recommendations for sliding resistance and working slabs will be required.

If retaining walls are required, consideration should be given to reinforced earth retaining structures.

Other concerns that in our opinion require clarification are:

- M.T.O. practice is to specify material parameters for granular backfill rather than provide equivalent fluid pressures.
- although deep-seated instability of the 8.5 m approach fills is not anticipated, the surficial stability of the high fill should be addressed.
- since groundwater elevations have been reported to be higher than proposed excavations and the non-cohesive strata are susceptible to disturbance, dewatering recommendations are required.
- recommendations for the proposed roadway protection scheme are required.
- recommendations for anticipated settlements under the approaches should be provided.

Our comments are intended only as suggestions for consideration. As you are aware our office was not involved in supervision of the geotechnical consultant and we would expect that any design recommendations would be their responsibility.

If there are any questions, please advise.

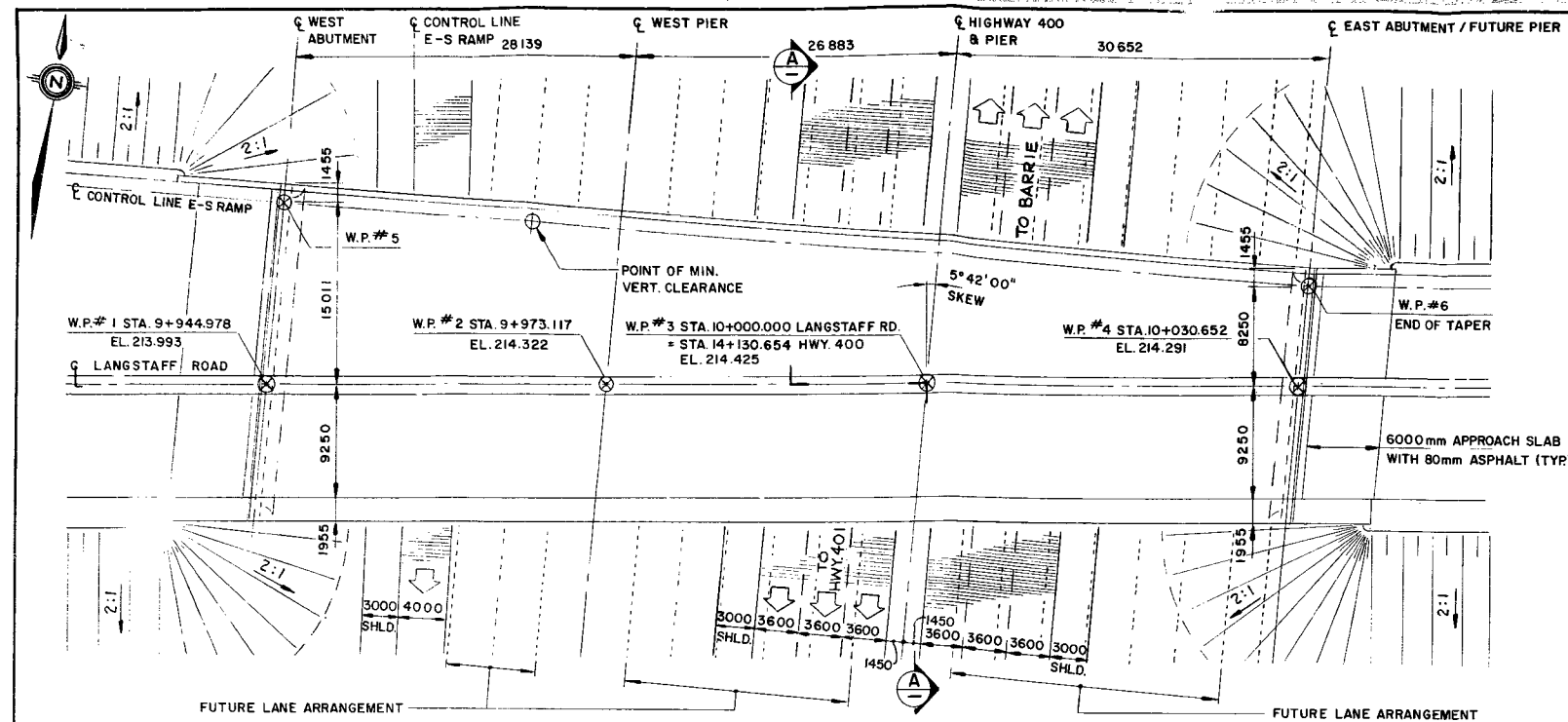

D. H. Dundas, P. Eng.
Sr. Foundation Engineer

for

M. Devata, P. Eng.
Chief Foundation Engineer

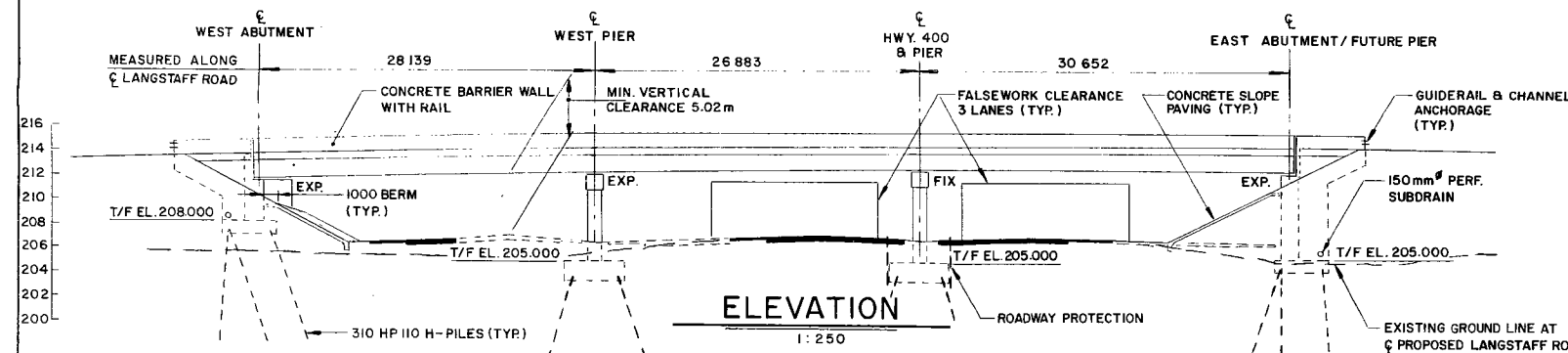
DHD/ms

c.c. - M. Holowka



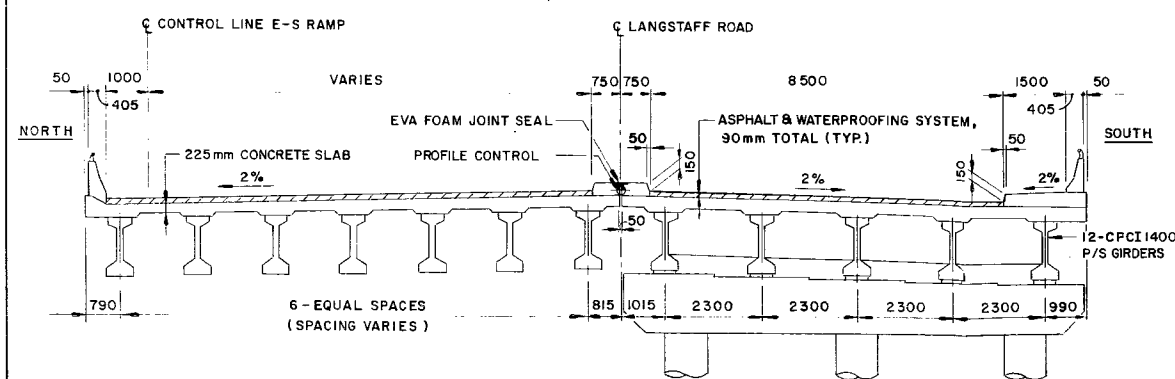
PLAN

1:250



ELEVATION

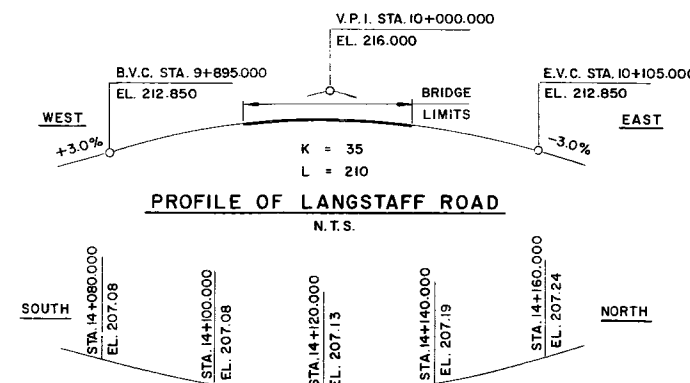
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SECTION A-A

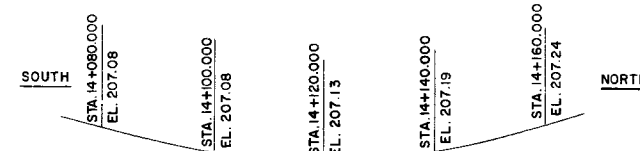
1:75

ALL HORIZONTAL DIMENSIONS \perp TO ϕ OF LANGSTAFF ROAD



PROFILE OF LANGSTAFF ROAD

N.T.S.



PROFILE OF HIGHWAY 400

CROWN OF N.B. & S.B. LANES
N.T.S.

GENERAL NOTES

- CLASS OF CONCRETE**
PRESTRESSED GIRDERS 40 MPa
FOOTINGS 30 MPa
REMAINDER 30 MPa
- CLEAR COVER TO REINFORCING STEEL**
FOOTINGS 100±25
ABUTMENTS AND WINGWALLS
FRONT FACE 80±20
BACK FACE 70±20
PIERS 80±20
DECK
TOP 70±20
BOTTOM 40±10
MEDIAN & SIDEWALK 70±20
REMAINDER 70±20 UNLESS OTHERWISE NOTED
- REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX C DENOTE COATED BARS.
- DESIGN LOADING - O.H.B.D.C. - A-83.
- STATION & ELEVATIONS ARE GIVEN IN METRES ALL OTHER DIMENSIONS ARE GIVEN IN MILLIMETRES.
- CONSTRUCTION NOTES**
o BEARING SEATS SHALL BE FINISHED LEVEL AND TO THE SPECIFIED ELEVATIONS.
o COMPACTED FILL, MAXIMUM GRAIN SIZE 75 mm SHALL BE PLACED UP TO THE BOTTOM OF FOOTING ELEVATION AT ABUTMENTS AND WEST PIER PRIOR TO DRIVING PILES.

LIST OF DRAWINGS

- GENERAL ARRANGEMENT
- BOREHOLE LOCATION & SOIL STRATA
- FOUNDATION LAYOUT
- WEST ABUTMENT LAYOUT
- EAST ABUTMENT LAYOUT
- WEST ABUTMENT WINGWALLS
- EAST ABUTMENT WINGWALLS
- WEST & EAST PIER LAYOUT
- WEST & EAST PIER REINFORCING
- GIRDER LAYOUT
- PRESTRESSED GIRDERS I
- PRESTRESSED GIRDERS II
- PRESTRESSED GIRDERS III
- DECK LAYOUT & ELEVATIONS
- DECK REINFORCING
- 6000 mm APPROACH SLAB
- BARRIER WALL WITH RAIL
- BARRIER WALL ON SIDEWALK
- CLASS I JOINT
- STANDARDS I
- STANDARDS II
- STANDARDS III
- STANDARDS IV
- ELECTRICAL
- QUANTITIES



MCCORMICK RANKIN
CONSULTING ENGINEERS

NO	DATE	REVISIONS	BY



ENGINEERING DEVELOPMENT DEPARTMENT

DESIGNED BY K.W.F.	CHECKED BY R.S.
DRAWN BY J.W.B.	CHECKED BY R.S.
DATE FEB. 1989	

TOWN OF VAUGHAN
HIGHWAY 400 UNDERPASS
AT LANGSTAFF ROAD
GENERAL ARRANGEMENT

DWG. NO. PI
CONT. NO.
SITE NO. 37-1298
SHEET NO.