Presented By John Anderson

3/17/82
Job 7220
Response to GEI 41, 42
20/135

THE DYNAMIC CONE PENETROMETER

A penetrometer is a device forced into the soil to measure its resistance to vertical penetration. In a dynamic penetration test, the penetrometer is driven into the soil by a hammer or falling weight. Soil penetrometers are used for qualitative measurements of relative density of cohesionless soils or consistency of cohesive soils. Penetrometers have been designed to give qualitative measurements of soil penetration resistance for correlation with soil physical properties such as relative density, unconfined compressive strength or shear strength, bearing value, or safe soil pressure.

Dynamic Resistance - The oldest and simplest form of soil penetrometers consists of driving a rod into the ground by repeated blows of a hammer. The penetration of the rod for a given number of blows with a hammer of constant weight and drop, or the number of blows required per foot penetration of a rod, may be used as an index of penetration resistance and correlated directly with local foundation experience. The numerical value of this index depends not only on the nature of the soil but also on the diameter, length, and weight of the rod in relation to the weight and drop of the hammer.

Cone penetration tests were developed as an easy and quick method for determining the approximate shearing resistance of noncohesive soils. The dynamic cone penetrometer consists of a 60-degree cone of steel attached to a section of rod. The rod is driven into the ground with a 10-pound drop hammer. The hammer is raised and allowed to fall a distance of 24 inches. The 60-degree cone is 1-1/8 inch in diameter. The diameter of the rod is smaller than that of the conical drive point, and short sections of rods are joined by couplings. This arrangement helps to reduce friction and permits use of a drive point and rod of smaller dimensions. When representative samples are desired of a certain strata, the drive point can be replaced with a small drive sampler. The weight of the entire equipment is about 25 pounds. The soil around and below the cone is slightly disturbed as the test progresses; therefore, the penetration does not correspond directly to the shearing resistance of the undisturbed soil. The penetration will also depend to some extent on the speed with which the cone is pushed into the soil. Despite these shortcomings, the cone penetrometer may be used advantageously in many soil investigations and is easier to perform than other more complicated field tests.

Variations in cone penetrometer resistance may indicate dissimilar soil layers and the numerical values of these resistances permit an estimation of some of the physical properties of the strata. The penetrometer can therefore be considered a method of both exploration and field testing. The advantages and limitations of this method may be summarized as follows.

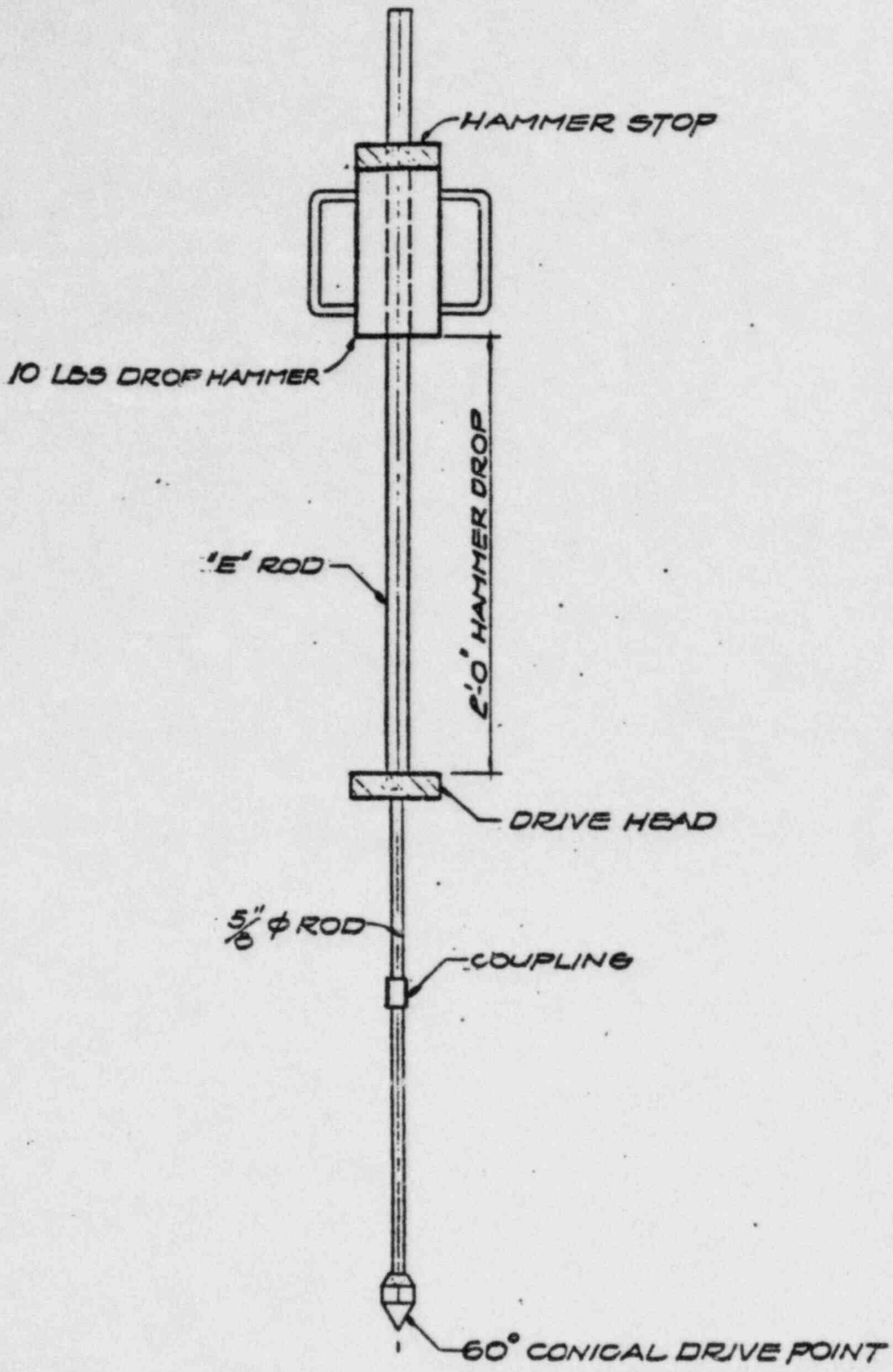
When the resistance to penetration is properly determined, the profiles obtained generally furnish consistent data on the depths of the different soil strata, but misleading results can also be obtained when the soil contains gravel and boulders. Profiles of continuous penetration resistance may indicate the presence of a thin layer which often remains unobserved in boring operations, but the strata encountered cannot be definitely identified by resistance to penetration alone. The cone penetrometer method is generally faster and less expensive than other more complicated methods.

Resistance to penetration profiles also indicates the consistency of cohesive soils and the compactness or relative density of cohesionless soils in situ. This information is valuable when undisturbed samples are difficult to obtain, as in saturated cohesionless soils, when many tests are required, or testing time is a factor. Generally, small and large areas can be explored rapidly and economically by penetrometer methods, especially when the depth of exploration is moderate and the soils are noncohesive.

The results of the cone penetrometer test should be used as indicators only. In comparing allowable bearing pressure with penetration resistance, the depth of confinement is critical in granular soil. Thus, correlations should be developed for each specific project. This correlation can be developed by using the pressure meter, field density tests (sand cone or nuclear), or other methods so that a given blow count can be related to a specific soil property, such as density or modulus.

3/17/82

Response to CEI 4.1.4.2



— SKETCH OF —

DYNAMIC CONE PENETROMETER

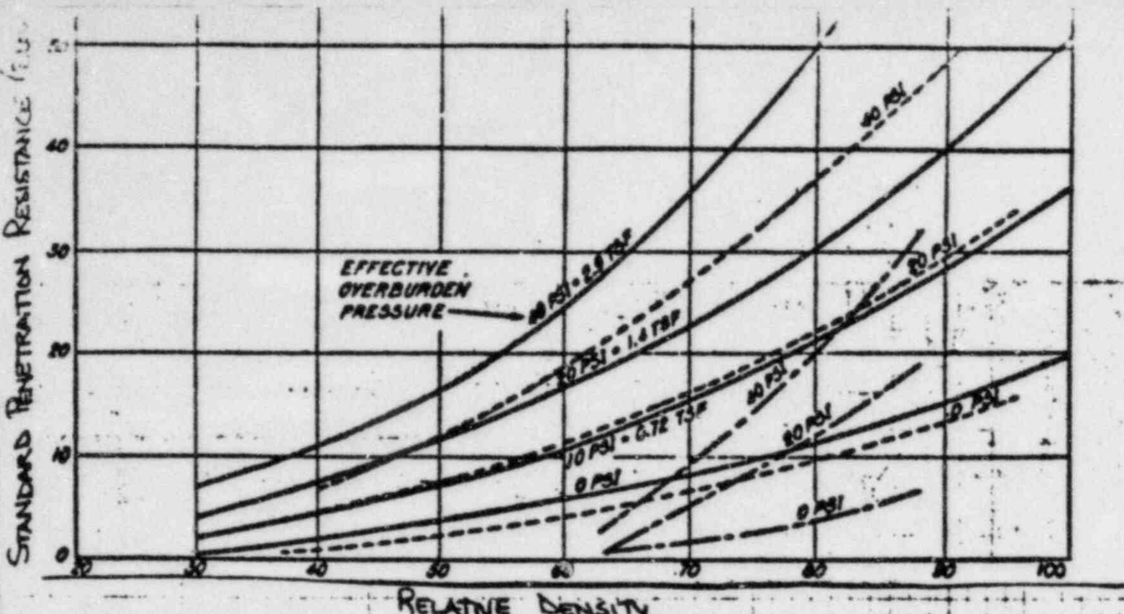
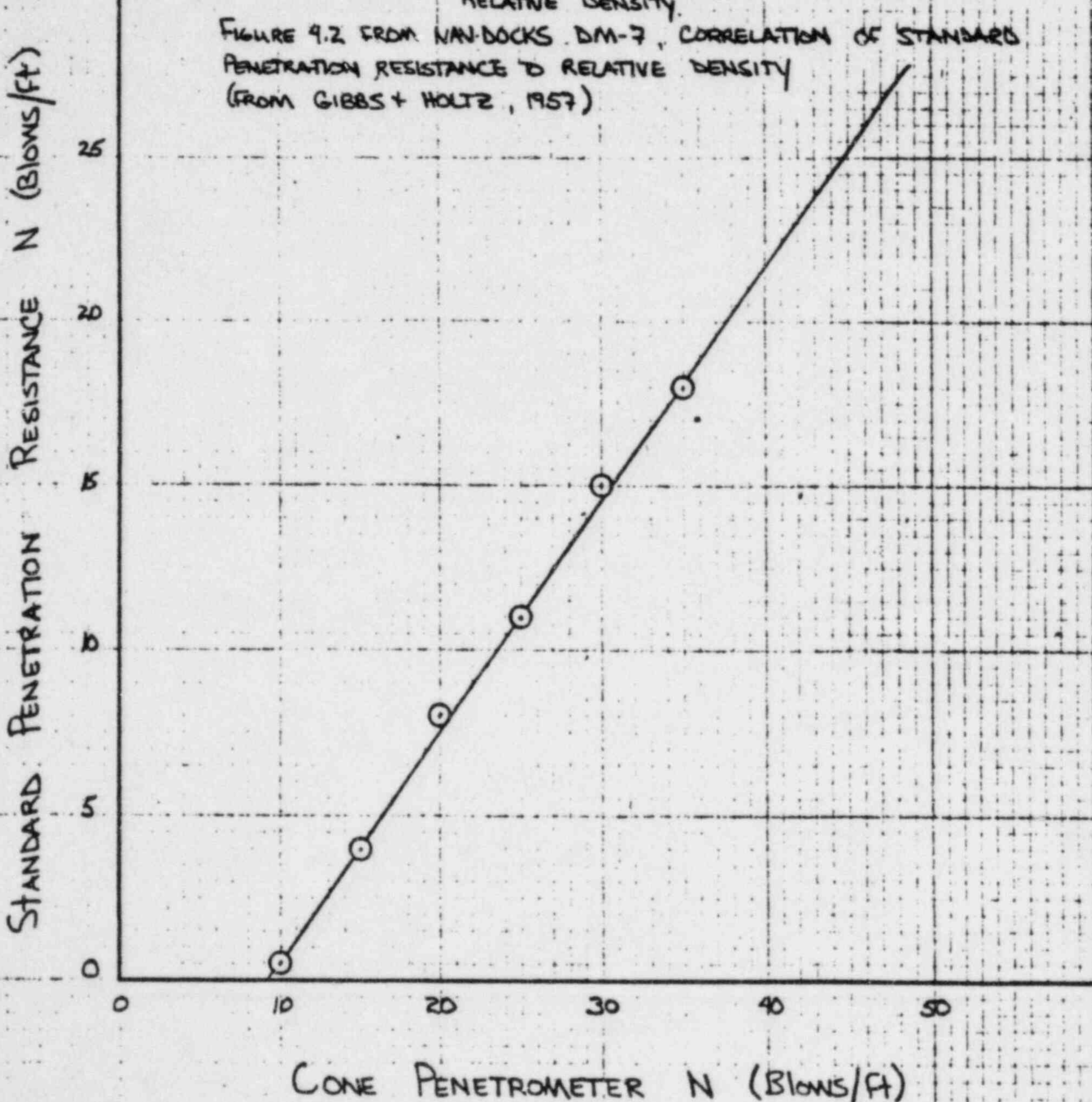


FIGURE 9.2 FROM NAV-DOCKS DM-7, CORRELATION OF STANDARD PENETRATION RESISTANCE TO RELATIVE DENSITY (FROM GIBBS + HOLTZ, 1957)



Rec'd 3/19/82 from J. Anderson

MUESER, RUTLEDGE, JOHNSTON & DESIMONE
CONSULTING ENGINEERS
FOR MIDLAND POWER PLANT -

SHEET No. A OF
FILE 5345A
MADE BY W.W. DATE 8-25-81
CHECKED BY ____ DATE ____

FORM 3

20/B5

MIDLAND PLANT UNITS 1 & 2

CONSUMER POWER COMPANY

SERVICE WATER INTAKE STRUCTURE

DESIGN CALCULATIONS

STUDY OF UNDERPINNING & SETTLEMENT ANALYSIS

DESIGNERS & CHECKERS	INITIALS
WALTER WALCHUK	WW
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JAMES P. GOULD	JPG

FOR INFORMATION ONLY

7220-C1003-2
SUB 2

P2

20/B5

SERVICE WATER INTAKE STRUCTURE
STUDY OF UNDERPINNING

STATEMENT OF PROBLEM:

PORTION OF THE SERVICE WATER INTAKE STRUCTURE IS SUPPORTED ON COMPACTED FILL. TO PREVENT POTENTIAL FUTURE SETTLEMENTS, DECISION WAS MADE TO UNDERPIN THIS PORTION OF THE STRUCTURE TO THE UNDERLYING TILL LEVEL.

DESIGN CRITERIA

THE PROPOSED UNDERPINNING IS TO BE DESIGNED TO TRANSMIT ALL BUILDING AND SEISMIC LOAD TO THE FIRM SUBGRADE LEVEL.

SOURCES OF DESIGN CRITERIA

BECHTEL INFORMATION DRAWINGS SK-C-748, REV A AND SK-C-749, REV. A OF MARCH 6, 1981, PLUS BECHTEL'S STRUCTURAL & PIPING DESIGN DRAWING AND AVAILABLE SOIL BEARING DATA AS OUTLINED IN THE "CONSUMERS POWER COMPANY" FINAL SAFETY ANALYSIS REPORT, VOLUME 4.

ASSUMPTIONS SUB 2

AS LISTED ON INDIVIDUAL PAGES OF CALCULATIONS.

20/85

METHOD OF ANALYSIS

CONVENTIONAL, WITH REFERENCES LISTED INDIVIDUALLY.

CALCULATIONS.

SEE INDEX OF CALCULATIONS, SHEET NO. C

SUMMARY, CONCLUSIONS & RECOMMENDATIONS

LISTED ON INDIVIDUAL CALCULATION SHEETS AND ON THE UNDERPINNING DRAWINGS U-1 & U-2.

SOURCES OF FORMULAS & REFERENCES

LISTED ON INDIVIDUAL CALCULATIONS SHEETS, UNLESS GENERALLY KNOWN & USED.

APPENDICES

NONE

SERVICE WATER INTAKE STRUCTURE

20/85

INDEX OF CALCULATIONS

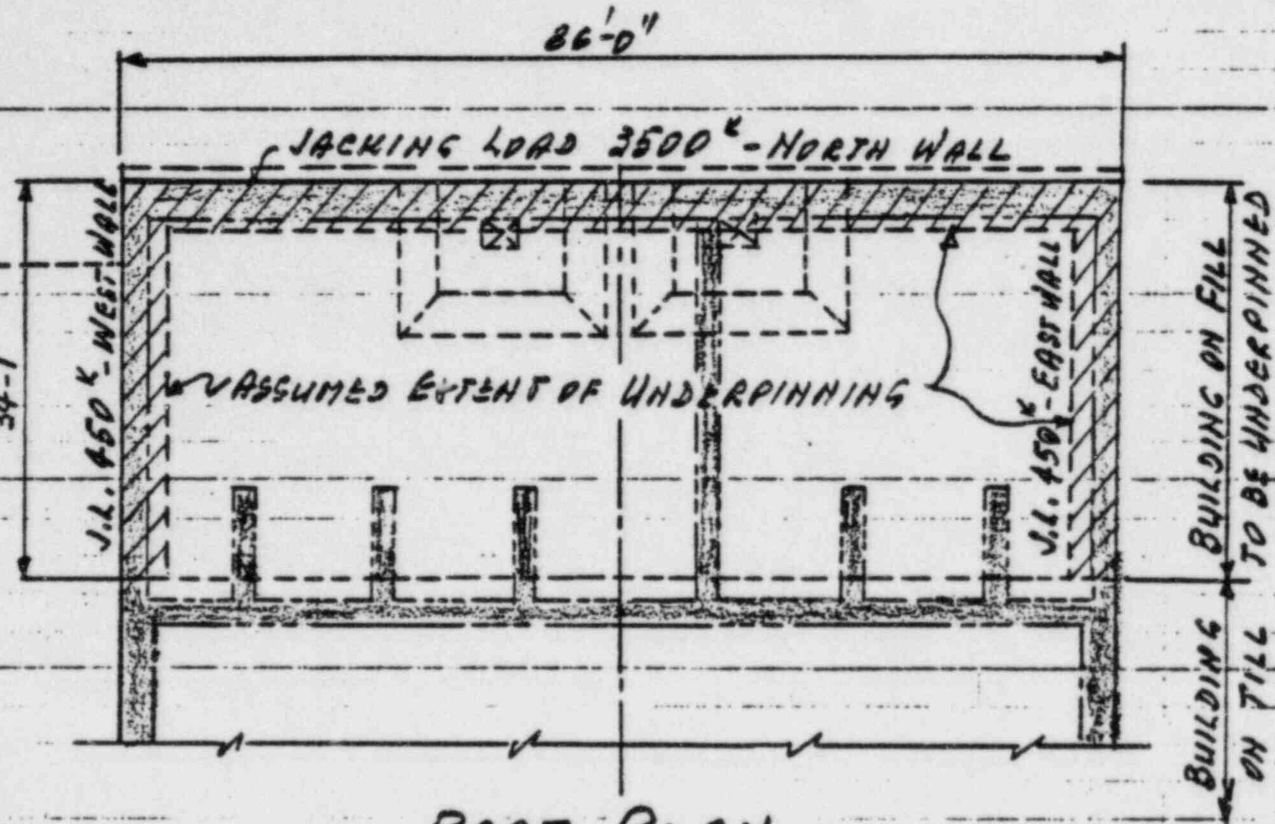
PLATE NUMBERS

SUB 2

20/85

SERVICE WATER INTAKE STRUCTURE

STUDY OF UNDERPINNING REQUIREMENTS



PART PLAN
1/16" = 1'-0"
(PIPING & EQUIPMENT NOT SHOWN)

DESIGN ASSUMPTIONS

1. TOTAL FINAL JACKING LOADS AS SHOWN ON PLAN
2. INITIAL JACKING LOAD = 2500k ON THE NORTH WALL AND 350k EACH ON THE EAST AND WEST WALLS
3. ULTIMATE BEARING CAPACITY OF TILL IS 52,000 LBS/SF. (SEE FINAL SAFETY ANALYSIS REPORT, VOL. 3 TABLE 2.5-14)

4. ALLOWABLE SOIL BEARING INTENSITY FOR THE DEAD AND LIVE LOADS, BASED ON A FACTOR OF SAFETY OF 3 IS $\frac{52}{3} = 17.3$ KSF MAXIMUM

5. ALLOWABLE SOIL BEARING INTENSITY FOR THE DEAD, LIVE AND EARTHQUAKE LOADS, BASED ON A FACTOR OF SAFETY OF 2 IS $\frac{52}{2} = 26.0$ KSF. MAXIMUM

6. FOLLOWING LOADS WILL BE CONSIDERED FOR ANALYSIS OF THE UNDERPINNING PIERS, TO SATISFY SOIL BEARING LIMITATIONS.

a) SUSTAINED LOADS (F.S. = 3.0) WILL INCLUDE;
1. FINAL JACKING LOADS
2. DOWNDRAG LOADS OF THE SURROUNDING FILL
3. DIFFERENTIAL WEIGHT OF PIER CONCRETE

b) TEMPORARY LOADS (F.S. = 2.0) WILL INCLUDE ITEMS 1, 2 & 3 FROM "a" PLUS EARTHQUAKE LOADS.

A) SUSTAINED VERTICAL LOADS AT SUBGRADE

1. FINAL JACKING LOAD ON THE NORTH WALL.

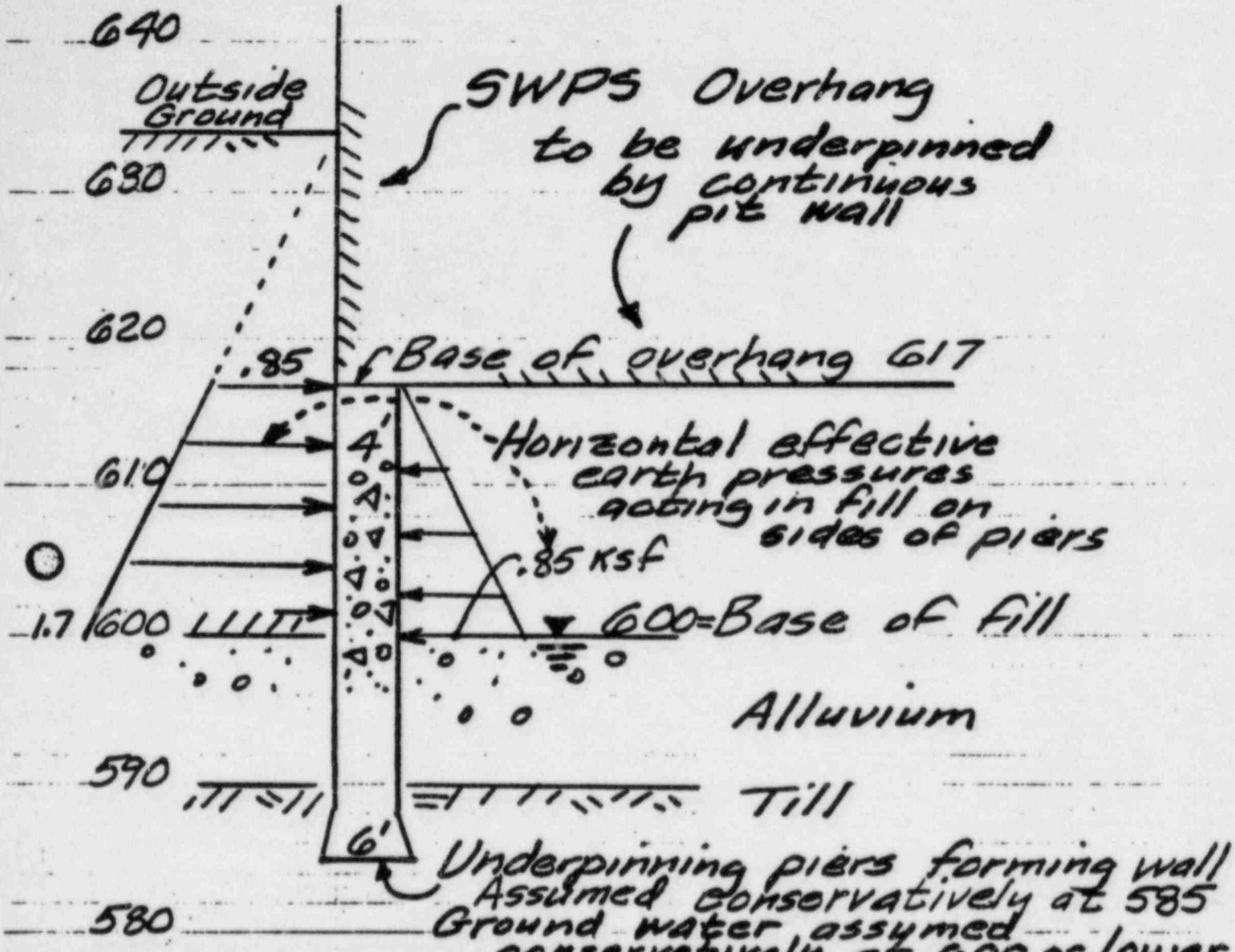
$$p_{N_4} = \frac{3500}{86 \times 4} = 10.2\ ^{K}/_{SF} \text{ ON A 4' WIDE BASE}$$

$$p_{N_6} = \frac{3500}{86 \times 6} = 6.8\ ^{K}/_{SF} \text{ ON A 6' WIDE BASE}$$

MUESER, RUTLEDGE, JOHNSTON & DESIMONE
CONSULTING ENGINEERS
FORM 2 For Midland SWPS Underpin
Made By JPG Date 11-24-81
Checked By W.H. Date 1-26-82

Evaluation of Downdrag Forces on Underpinning Piers

20/85



For downdrag computation, assume horizontal pressure based on 50 psf/ft equivalent fluid pressure with drawdown to below the base of fill. Downdrag is produced only by side shear on fill/pier interface.

Assume the fill is settling fairly actively during underpinning and initial plant drawdown, but rate of settlement slow

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FORM 3 FOR Midland SWPS Underpin
MADE BY JPG DATE 11-24-81
CHECKED BY G.G. DATE 1-26-82

Downdrag Forces on Underpinning Piers

20/85

On outside of pier: psf/ft

σ_h at 617 = 17' × 50 = .85 ksf

σ_h at 600 = 34' × 50 = 1.7 ksf

On inside of pier:

σ_h at 617 ≠ 0 ksf

σ_h at 600 = .85 ksf

Resultant horizontal force: per lf of pier

Inside + outside

(.85) 17'/2 = 7K

$\left[\frac{.85+1.7}{2}\right]$ 17' = 22K Total = $\underline{29^K}$

Assume a friction factor on fill/pier surface = 0.6 (very conservative - but give it a try)

Total shear force = 0.6 × 29^K ≅ 18^K

Divide over 6' wide bearing = 18/6 = 3.0 Ksf

Now assume all of soil weight above bell caves on bell side:

This would equal 1 foot-wide column on each side of pier: kcf

[49' outside + 32' inside] × .130 = 10.5^K

Divide over 6' wide bearing = $\frac{10.5}{6}$

= 1.7 ksf

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Therefore peak drag on pier sides and bell = 1.7 + 3.0 = $\underline{4.7 \text{ ksf}}$ ← Peak drag

Long-term drag assume that drag is about 1/10 of peak = 0.3 ksf on base

Add weight on bell: 1.7 + 0.3 = $\underline{2 \text{ ksf}}$ < long term

SUB 2

2. DOWNDRAG LOAD OF THE SURROUNDING FILL ; SEE SHEET NO'S 2a & 2b

$p_{d_4} = \frac{18}{4} = 4.5$ KSF ON A 4' WIDE BASE

$p_{d_6} = \frac{18+10.5}{6} = 4.7$ KSF ON A 6' WIDE BASE INCLUDING WEIGHT OF SOIL ABOVE THE BELL SIDES.

3. DIFFERENTIAL WEIGHT OF CONCRETE

$p_{c_4} = (150-125) \times 30 \cong 0.8$ KSF ON A 4' WIDE BASE

$p_{c_6} = 0.8 \times 4/6 \cong 0.5$ KSF ON A 6' WIDE BASE

COMBINED SUSTAINED BEARING INTENSITY AT BASE OF THE NORTH WALL UNDERPINNING PIERS

$\Sigma P_{s_4} = 10.2 + 4.5 + 0.8 = 15.5$ KSF ON A 4' WIDE BASE

$\Sigma P_{s_6} = 6.8 + 4.7 + 0.5 = 12.0$ KSF ON A 6' WIDE BASE

NOTE: UNDER THE LONG TERM CONDITION THE DOWNDRAG LOAD MIGHT BE ASSUMED AT 2.0 KSF, FURTHER REDUCING THE SUSTAINED BEARING INTENSITY AT BASE OF WALL.

EFFECTS ON THE EARTHQUAKE LOADING ON THE BEARIG INTENSITY AT BOTTOM OF WALL.

7220-C100-3-2

$p_{s_4} = \frac{38.4}{4} = 9.6$ KSF ON A 4' WIDE BASE

$p_{s_6} = \frac{38.4}{6} = 6.4$ KSF ON A 6' WIDE BASE

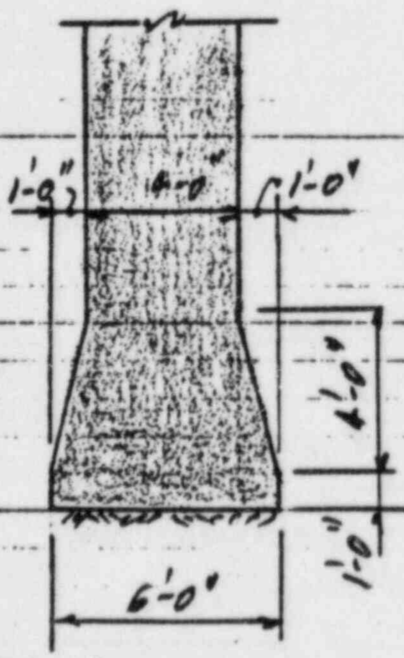
SUB 2
7/10

COMBINED TEMPORARY BEARING INTENSITY AT BASE OF THE NORTH WALL UNDERPINNING INCLUDING SEISMIC LOADING.

$\Sigma P_{E4} = 10.2 + 4.5 + 0.8 + 9.6 = 25.1$ KSF ON A 4' WIDE BASE

$\Sigma P_{E6} = 6.8 + 4.7 + 0.5 + 6.4 = 18.4$ KSF ON A 6' WIDE BASE

NOTE: TO PERMIT FOR REASONABLE TOLERANCES DURING CONSTRUCTION AND POTENTIAL MINOR LOCAL VARIATIONS IN THE QUALITY AND UNIFORMITY OF THE SOIL BEARING STRATA IT IS RECOMMENDED TO INCREASE BASE OF THE UNDERPINNING PIERS FROM 4 TO 6 FEET AT THE NORTH WALL.



BASE OF UNDERPINNING AT NORTH WALL

7220-C100-3-2
SUB 2
P11

MUESER, RUTLEDGE, JOHNSTON & DESIMONE
CONSULTING ENGINEERS
FOR MIDLAND POWER PLANT - UNDERPINNING

MADE BY N.W. DATE 3-18-81
CHECKED BY SKS DATE 3-19-81
LH. 2-1-82

FORM 3

MAXIMUM SOIL BEARING INTENSITY AT BASE OF UNDERPINNING AT THE EAST & WEST WALLS

A) DUE TO SUSTAINED VERTICAL LOADS

a) FINAL JACKING LOADS: $p_J = \frac{450}{4 \times 30} = 3.8$ KSF

b) DOWNDRAG $p_D = \frac{18}{4} = 4.5$ KSF.

c) DIFFERENTIAL WEIGHT OF CONCRETE: $p_C = 0.8$ KSF.

$\Sigma P_S = 3.8 + 4.5 + 0.8 = $ **9.1 KSF O.K.**

B) DUE TO TEMPORARY LOADING INCLUDING SAFE SHUTDOWN EARTHQUAKE LOADING.

NOTE: LOADS a, b & c ARE SAME AS ABOVE:

d) SSE EARTHQUAKE: $p_E = 10$ K/LIN.FT

$\Sigma P_E = 9.1 + \frac{10}{4} = $ **11.6 KSF O.K.**

NOTE: ALL OF THE ABOVE SOIL BEARING INTENSITIES ARE WELL WITHIN THE ACCEPTABLE LIMITS.

SUB 2

7220-C100-3-2 P12

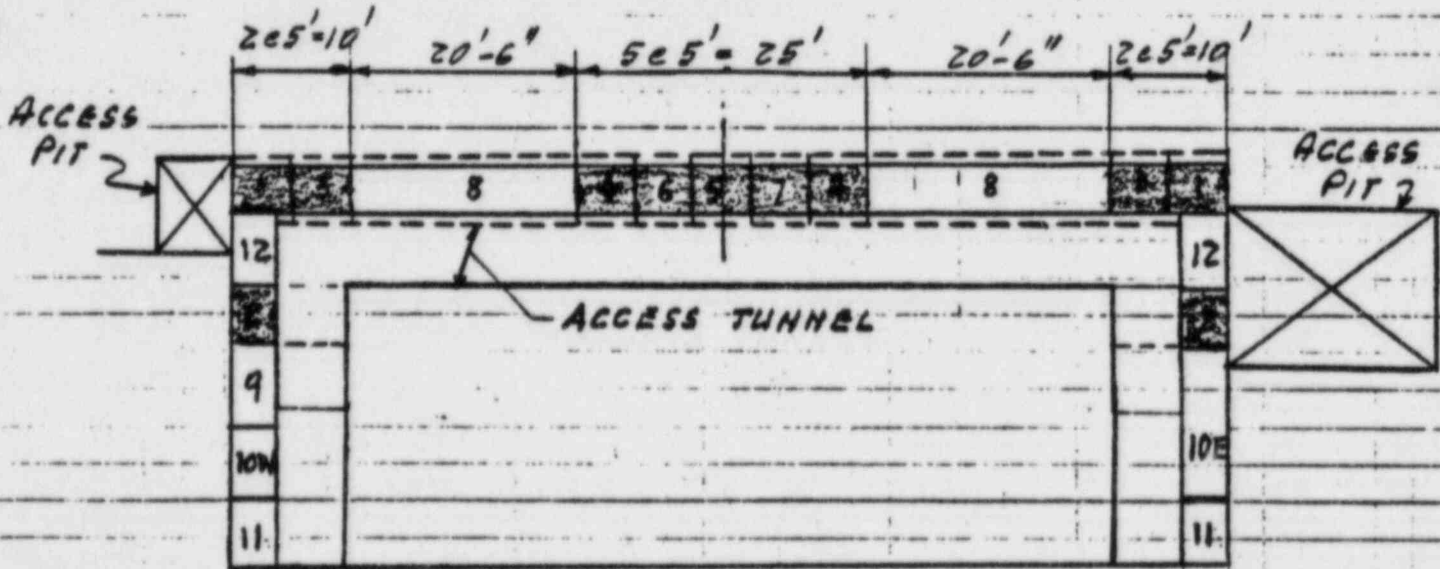
INITIAL STAGES OF UNDERPINNING

INITIAL JACKING LOAD AT THE NORTH WALL = 2500^{K}

OR $\frac{2500}{86} = 29.1$ K/LIN. FT.

ASSUMING PROPORTIONAL REDUCTION FOR THE EAST AND WEST WALL, THE INITIAL JACKING LOAD ON THESE WALLS WOULD BE: $\frac{2.5}{3.5} \times \frac{450}{30.1} = 10.7$ K/LIN FT.

TRY 3 - 5'-0" WIDE PIERS AT THE NORTH-EAST AND NORTH-WEST CORNERS AND 5 - 5'-0" PITS AT CENTER OF THE NORTH WALL AS SHOWN BELOW;



PLAN

INITIAL UNDERPINNING PIERS

7220-C100-3-2 A3

NOTE: UNDERPINNING PIERS SHALL BE CONSTRUCTED IN A SEQUENCE SHOWN ABOVE, SECTIONS MARKED 8, 9 & 10 WOULD BE CONSTRUCTED AS COMPLETE UNITS.

SUB 2

MUESER, RUTLEDGE, JOHNSTON & DESIMONE
CONSULTING ENGINEERS
FOR MIDLAND POWER PLANT-UNDERPINNING

MADE BY W.W. DATE 3-19-81
CHECKED BY SKS DATE 3-20-81
L.H. 2-1-82

TEMPORARY BEARING INTENSITY AT THE CORNER PIERS ① & ③, BASED ON THE INITIAL JACKING LOAD OF 2500^k ON THE NORTH WALL & PROPORTIONALLY REDUCED JACKING LOADS ON THE EAST AND WEST WALLS, INCLUDING DIFFERENTIAL WEIGHT OF PIER CONCRETE AND SOIL DRAG FORCES ON THE COMPLETED PIERS PLUS PORTIONS OF THE THEORETICAL JACKING LOADS FROM THE ADJOINING AREAS UNDER CONSTRUCTION:

a) JACKING LOAD ON PIERS 1 & 3; $29.1 \times 1/6$ = 4.9 KSF.
b) DIFF. WEIGHT OF CONC. $0.8 \times 4/6$ = 0.5 - " -
c) DRAG FORCES ON PIERS 1 & 3 = 4.7 - " -
d) EARTHQUAKE = 0.0 - " -
e) FROM THE ADJOINING NORTH WALL
$29.1 \times 10.25 \times 1/6 \times 1/10$ = 5.0 - " -
f) FROM THE ADJOINING EAST (WEST) WALL
$10.7 \times 6.0 \times 1/6 \times 1/10 \times 1/2$ = 0.6 - " -

$p_{i_{1-3}} = (4.9 + 0.5 + 4.7 + 5.0 + 0.6)$ = **15.7 KSF O.K.**

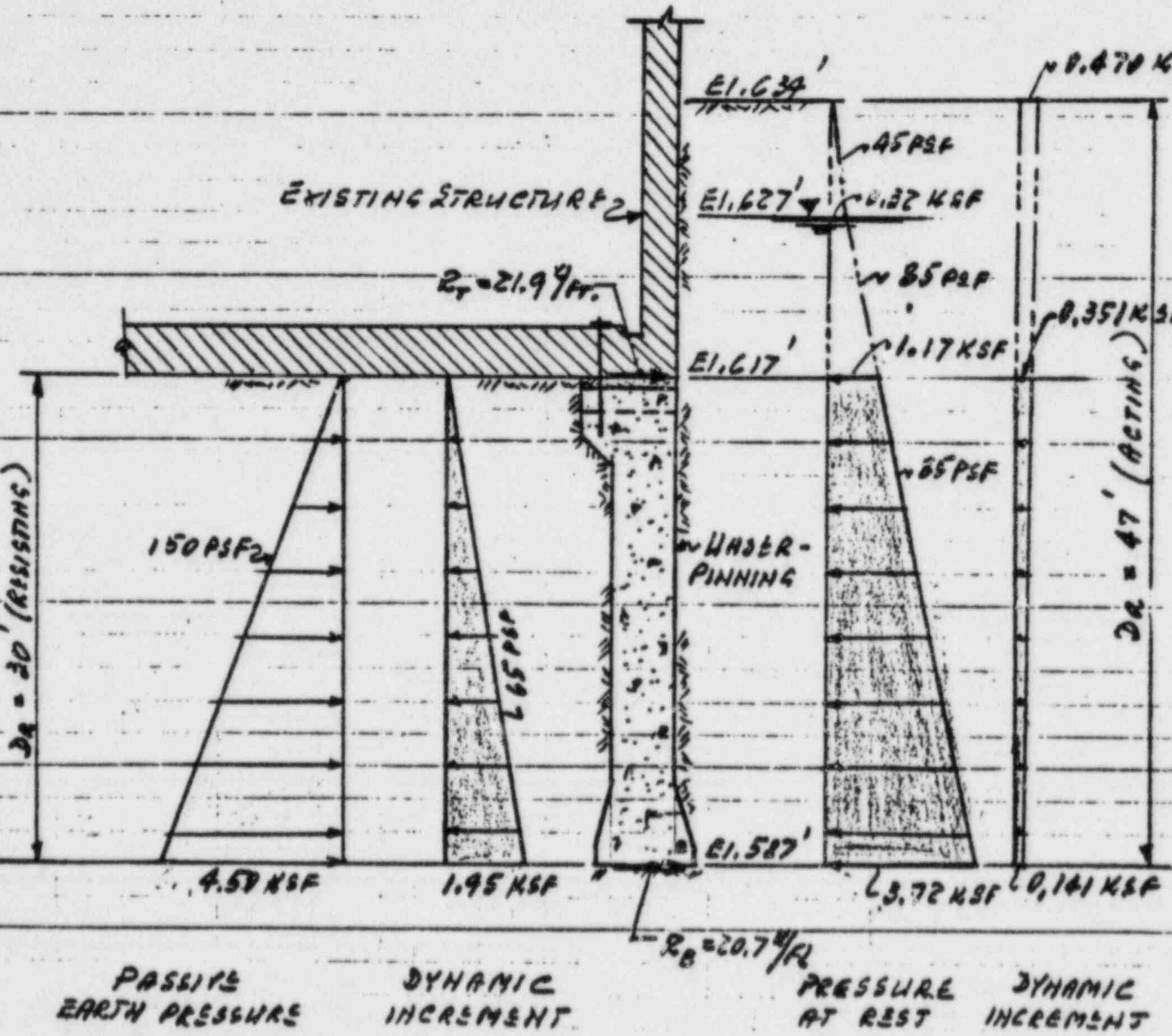
NOTE: SOIL BEARING INTENSITY AT THE INITIAL 5 CENTER PIERS AT THE NORTH WALL WILL NOT GOVERN SINCE SOMEWHAT LARGER BASE AREA IS AVAILABLE:

SUB 2

FORM 2

INVESTIGATION OF STATIC AND DYNAMIC HORIZONTAL LOADING ON THE UNDERPINNING PIERS

NOTE: STATIC AND DYNAMIC EARTH PRESSURES ARE BASED ON FSAR FIGURE 2.5-45 AND TABLE 2.5-15



TYPICAL SECTION - NORTH WALL

7220-C100-3-2

NOTE: ASSUME GROUND WATER LEVEL OUTSIDE AT El. 627'

P15

SUB 2

FORM 3

COMPACTED FILL ABOVE ELEVATION 600'± CONSISTS OF SANDY & CLAYEY SEAMS. FILL BELOW EL 600'± IS PRIMARILY ALLUVIUM. ASSUME CONSERVATIVELY ACTIVE PRESSURE FOR THE FULL DEPTH OF FILL EQUAL TO AN AVERAGE PRESSURE OF SANDY & CLAYEY LAYERS.

a) ABOVE GROUND WATER LEVEL ; p_{av_d} = 45 PSF
b) BELOW -"- -"- -"- p_{av_s} = 85 -"-

CONFIGURATIONS OF THE ACTIVE AND PASSIVE EARTH PRESSURES AND VALUES OF THE SEISMIC DYNAMIC INCREMENTS ARE BASED ON FSAR FIG. 2.5-45 STATIC PRESSURE VALUES ARE OBTAINED FROM TABLE 2.5-15.

PASSIVE RESISTANCE : p_p = 150 PSF (SUBMERGED)

ACTIVE PRESSURES:

AT ELEV. 617' ; $p_a = (7 \times 0.045) + (10 \times 0.085) = \underline{1.17\ KSF}$.

-"- -"- 587' ; $p_a = 1.17 + (30 \times 0.085) = \underline{3.72\ KSF}$

DYNAMIC INCREMENT ON THE ACTIVE SIDE:

AT EL. 634' $p_{634} = 10D = 10 \times 47 = 470\ PSF = 0.47\ KSF$

AT EL. 587 $p_{587} = 3D = 3 \times 47 = 141\ PSF = 0.141\ KSF$

AT EL. 617 $p_{617} = 0.141 + (0.47 - 0.141) \times \frac{30}{47} = 0.351\ KSF$.

7220-C100-3-2

SUB 2 P16

PASSIVE RESISTANCE

AT ELEV. 587' $p_p = 0.150 \times 30 = 4.50$ KSF

DYNAMIC INCREMENT ON PASSIVE SIDE

AT. EL. 587' $p_{D_i} = -0.065 \times 30 = -1.95$ KSF.

SUMMATION OF ACTIVE PRESSURES:

$\Sigma P_{A_H} = (1.17 + 3.72) \times 1/2 \times 30 + (0.351 + 0.141) \times 1/2 \times 30 =$
$= 73.4^k + 7.4 = \underline{80.8\ ^k/ft.}$

SUMMARY OF RESISTING PRESSURES

$\Sigma P_{R_H} = (4.50 \times 30 \times 1/2) - (1.95 \times 30 \times 1/2) =$
$= 67.5^k - 29.3 = \underline{38.2^k}$

$\Sigma P_A - \Sigma P_R = 80.8 - 38.2 = \underline{42.6\ ^k/ft}$

NOTE: THE DIFFERENCE BETWEEN TOTAL ACTING AND PASSIVE RESISTING PRESSURES WILL BE RESISTED BY FRICTION BETWEEN THE TOP OF PIERS AND EXISTING STRUCTURE AND BETWEEN BOTTOM OF PIER AND THE SUBGRADE.

FORM 8

MOMENTS ABOUT TOP OF PIER (TOP & BOTTOM HINGED)

ACTING MOMENTS:

$M_{T_A} = (1.17 \times 30^2 \times 1/6) + (3.72 \times 30^2 \times 1/3) + (0.351 \times 30^2 \times 1/6)$
$+ (0.141 \times 30^2 \times 1/3) = 175.5 + 1116 + 52.7 + 42.3 =$
≈ 1387 'K/FT.

RESISTING MOMENTS

$M_{T_R} = (4.50 \times 30^2 \times 1/3) - (1.95 \times 30^2 \times 1/3) = 1350 - 585$
$= 765$ 'K/FT

$\Delta M = 1387 - 765 = 622$ 'K/FT

$R_B = \frac{622}{30.} = \underline{20.7\ K/FT}$; $R_T = 42.6 - 20.7 = \underline{21.9\ K/FT.}$

FRICTION COEFFICIENTS AT TOP & BOTTOM OF PIERS.

FINAL JACKING LOAD AT TOP OF PIER ;

$V_J = \frac{3500}{86} = 40.7$ K/LIN FT.

FRICTION COEFFICIENT : $F_{R_T} = \frac{21.9}{40.7} = \underline{0.54 < 1.0\ O.K}$

NOT COUNTING THE CAPACITY OF 2 3/4" DIAMETER DOWELS, SPACED AT 3'-9" O.C. MAXIMUM.

BEARING INTENSITY AT BOTTOM OF PIER.

$V_B = 40.7 + 0.5 = 41.2$ K/FT MIN. (NEGLECTING DOWNDRAG)

FRICTION CAPACITY AT BOTTOM OF THE UNDERPINNING.

COHESION 730 PSF — USE 2/3 OF THIS VALUE

$F_c = 2/3 \times 0.730 \times 6 = 2.9$ K/LIN.FT.

FRICTION : $\phi = 36°$: USE FRICTION COEFFICIENT EQUAL TO $tg\ 2/3\phi = tg\ 24° = 0.445$

$R_{F_B} = (41.2 \times 0.445) + 2.9 = \underline{21.2\ K/Ft > 20.7\ K/Ft}$

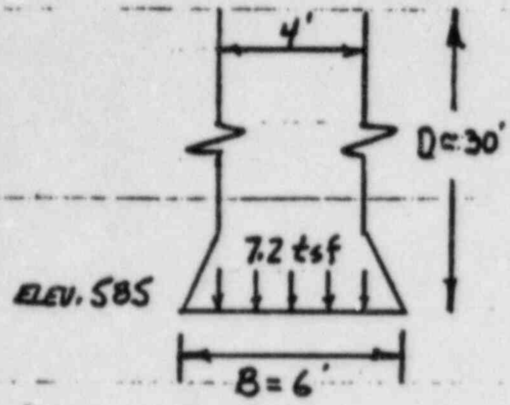
NOTE: THIS VALUE IS BASED ON VERY CONSERVATIVE ASSUMPTIONS. THE INCREASED RESISTANCE CAPACITY OF ALLUVIUM STRATA BELOW ELEVATION 600 AND RESISTANCE OF TILL WERE TOTALLY NEGLECTED. GROUND WATER LEVEL WAS ASSUMED AT THE HIGHEST POSSIBLE LEVEL AND WEIGHT OF SOIL ABOVE THE PIER PROJECTION AS WELL AS DOWNDRAG LOADS WERE NEGLECTED.

MUESER, RUTLEDGE, JOHNSTON & DESIMONE
CONSULTING ENGINEERS
FOR Service Water Building

Made By CHF Date 8-24-81
Checked By DHE Date 8-25-81

Form 2

SETTLEMENT OF UNDERPINNING PIER



L = length of wall = 86'

B = width of wall = 6'

$L/B = 86/6 = 14.33$

COMBINED JACKING LOAD + DOWNDRAG = 7.2 tsf = q

① REFERENCE: NAVFAC DM-7, fig. 11.9 & fig 4.4

δ_{vo} = settlement for D = 0 (i.e. base of pier @ ground surface)

$$\left.\begin{aligned} \delta_{vo} &= \frac{1.05\, q\, B\, C_s \sqrt{L/B}}{k_{v1}} \\ E_s &= 0.95\, k_{v1}\, (1-\nu^2) \end{aligned}\right\} \delta_{vo} = \frac{q\, B\, C_s \sqrt{L/B}}{\frac{E_s}{(1-\nu^2)}}$$

ν — Poissons ratio

C_s = SHAPE COEFFICIENT = 0.65

for D > 0, SETTLEMENT = $\delta_{vD} = \delta_{vo} \times \frac{\delta_{vd}}{\delta_{vo}}$

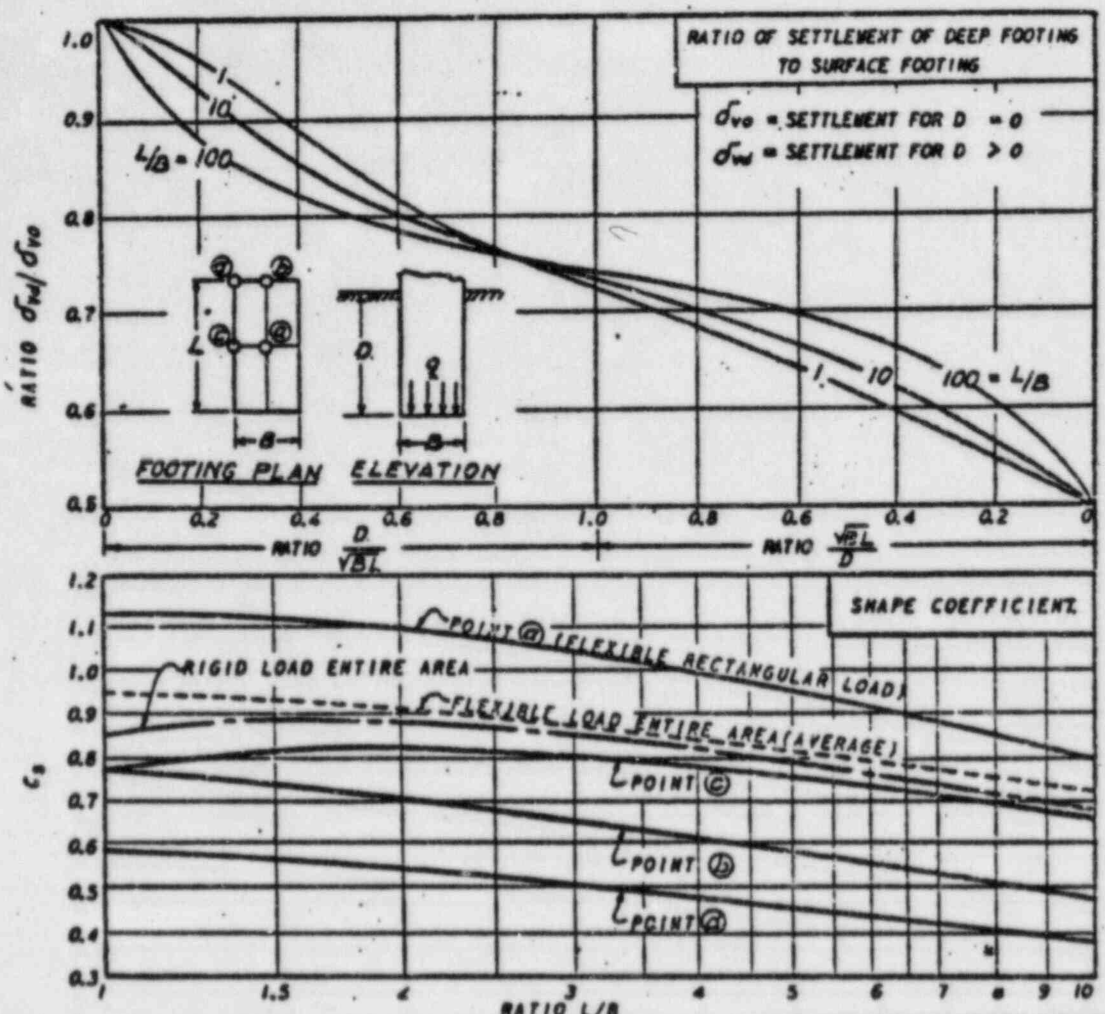
$$\therefore \frac{\sqrt{BL}}{D} = \frac{\sqrt{86 \times 6}}{\sim 30} = \cancel{0.76} \quad \text{ok} \longrightarrow \frac{\delta_{vD}}{\delta_{vo}} = 0.7$$

$$\boxed{\therefore \delta_{vD} = \frac{0.7\, q\, B\, C_s \sqrt{L/B}\, (1-\nu^2)}{E_s}}$$

7220-C100-3-2 P2B

SUB 2

NAVFAC DM-7



STIFF TO HARD FINE GRAINED SOILS
(MODULUS E_s IS CONSTANT WITH DEPTH)

SETTLEMENT FOR D = 0 (δ_{vo}):

$$\delta_{vo} = \frac{.75\, q\, B\, C_s \sqrt{L/B}}{E_s}$$

AND SINCE $E_s = .72\, K_{v1}$ (TONS AND FT. UNITS)

$$\delta_{vo} = \frac{1.05\, q\, B\, C_s \sqrt{L/B}}{K_{v1}}$$

SETTLEMENT FOR D > 0 (δ_{vd}):
COMPUTE δ_{vo} AND OBTAIN RATIO OF δ_{vd}/δ_{vo} FROM UPPER DIAGRAM.

POSITION OF GROUND WATER LEVEL DOES NOT AFFECT IMMEDIATE SETTLEMENT UNLESS SOIL IS PERMITTED TO SWELL. FOUNDATION MATERIAL IS ASSUMED UNIFORM TO A DEPTH OF AT LEAST 1.5B BELOW BASE OF FOOTING.

CONVERSION OF CONSTANT MODULUS E_s TO EQUIVALENT MODULUS E_s VARYING LINEARLY WITH DEPTH FOR APPLICATION TO LATERALLY LOADED PILES IN STIFF TO HARD CLAYS.

PROCEDURE:

1. GIVEN A VALUE OF CONSTANT E_s, ASSUME A VALUE OF f.
2. COMPUTE DEPTH Z TO POINT OF ZERO DEFLECTION OF PILE FROM FIGS. 13-4 OR 13-5.
3. RECOMPUTE f SO THAT VALUE OF $fz = 2E_s$.
4. RECOMPUTE DEPTH Z WITH REVISED f AND ALTER f AGAIN SO THAT $fz = 2E_s$.

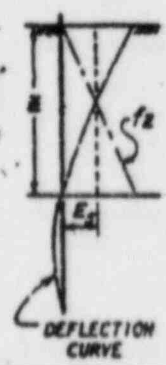


FIGURE 11-9
Immediate Settlement of Isolated Footings on Stiff to Hard Fine Grained Soils

SUB 2

7220-C100-3-2

$$\delta_{VD} = \frac{0.7 \times (7.2 \times 2) \times 6 \times 0.65 \times \sqrt{14.3}\ (1-\nu^2)}{E_s} = \frac{149\ (1-\nu^2)}{E_s}$$

$(E_s)_{AVG} = 3{,}600$ ksf

$E \pm 40\%$

COMPUTE SETTLEMENT for $E = 2{,}200,\ 3{,}600,\ 5{,}000$ ksf

& $\nu = 0.2,\ 0.33,\ 0.5$

SETTLEMENT (INCHES)

POISSONS RATIO (ν)	MODULUS OF ELASTICITY (ksf)		
	2200	3,600	5000
0.2	0.7"	~~0.4"~~	0.3"
0.33	0.6"	0.4"	0.3"
0.5	0.5"	0.3"	0.3"

This is the computation of total long-term settlement from elastic theory. From this we conclude the long-term value is probably 0.4 to 0.5"

MOST LIKELY SETTLEMENT.

$\boxed{\text{LIKELY SETTLEMENT} = 0.4" \pm 0.2"}$

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CONSULTING ENGINEERS
FOR Service Water Building

FORM 5

File 5345
Made By CHF Date 3-20-81
Checked By PHE Date 8-26-81

SETTLEMENT OF UNDERPINNING PIER

(2) USE SOIL PROFILE IN RESPONSE TO QUESTION 41 (page 2 of 6)

LAYER	ELEV. ft	THICKNESS ft	E ksf
A	585 to 582.5	2.5	2,400
B	582.5 to 562	20.5	3,600
C	562 to 543	19	4,800
D	543 to 503	40	4,800
E	503 to 363	140	4,800

E < (from reply to Question 41)

Not checked

COMPUTE SETTLEMENT

LAYER	DEPTH TO MIDLAYER ft (z)	$\frac{z}{\frac{1}{2} \times B}$ = z/3	Influence factor I	STRESS @ MIDLAYER = 7.2×2×I (ksf)	SETTLEMENT INCHES = $\frac{\sigma}{E} \times H \times 12$
A	1.3	0.4	~0.97	14.0	0.2
B	12.8	4.3	~0.29	4.2	0.3
C	32.5	10.8	~0.1	1.4	0.1
D	62	21	—	—	
E	152	51	—	—	

Σ 0.6"

This is an alternative calculation of total settlement by elastic methods, using E values varying with depth. It more or less confirms the range of 0.4 to 0.5".

7220-C100-3-2

23

SUB 2

SHEET NO. 12 OF 22

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FOR Service Water Building

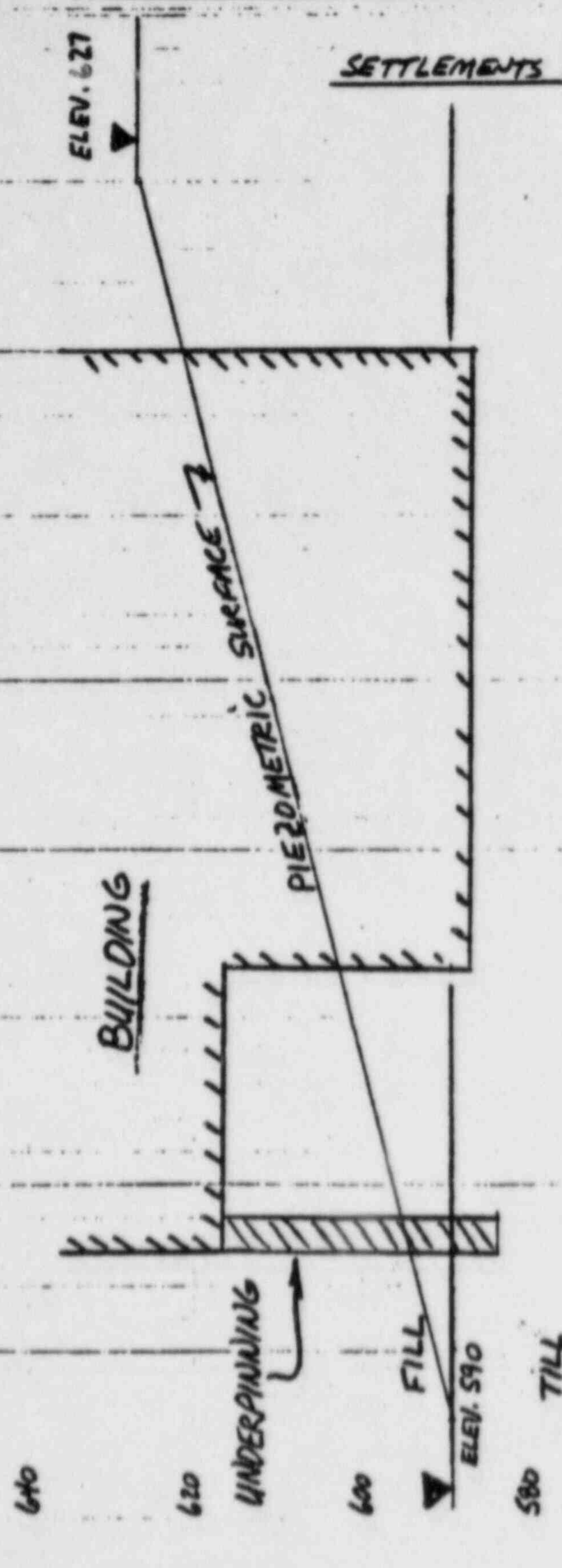
FILE 5345

MADE BY CHF DATE 8-24-81

CHECKED BY ______ DATE ______

FORM 8

SETTLEMENTS FROM GROUND WATER PUMPING



- GEOMETRY -

SETTLEMENTS from groundwater pumping

7220-C100-3-2

P27

SUB 2

SETTLEMENT FROM GROUND WATER PUMPING

Reference: Cedergren, "Seepage, Drainage, and Flow Nets", John Wiley & Sons, 1967

Casagrande, "Seepage Thru Dams", Harvard Graduate School of Engineering publication no. 209, 1937.

Use flow net to estimate water pressures below the Service Water Building

Seepage Principles 109

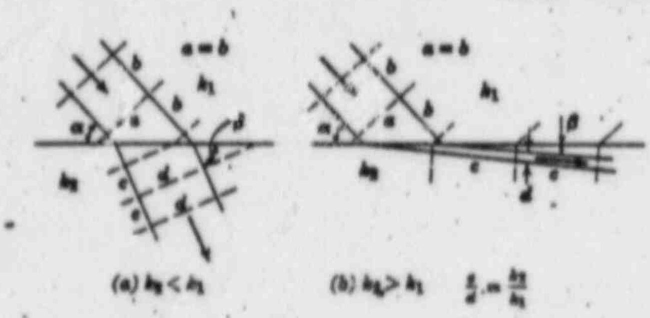


FIG. 3.10 Transfer conditions at boundaries between soils of differing permeabilities. (After A. Casagrande, *Seepage through Dams*, 1937.)

from Cedergren

The way flow lines deflect when they cross boundaries between soils of different permeabilities is shown in Fig. 3.10. The flow lines bend to conform to the following relationship:

$$\frac{\tan \beta}{\tan \alpha} = \frac{k_1}{k_2} \tag{3.10}$$

Simultaneously, the areas formed by the intersecting lines either elongate or shorten, depending on the ratio of the two permeabilities, according to the following relationship:

$$\frac{c}{d} = \frac{k_2}{k_1} \tag{3.11}$$

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FOR Service Water Building

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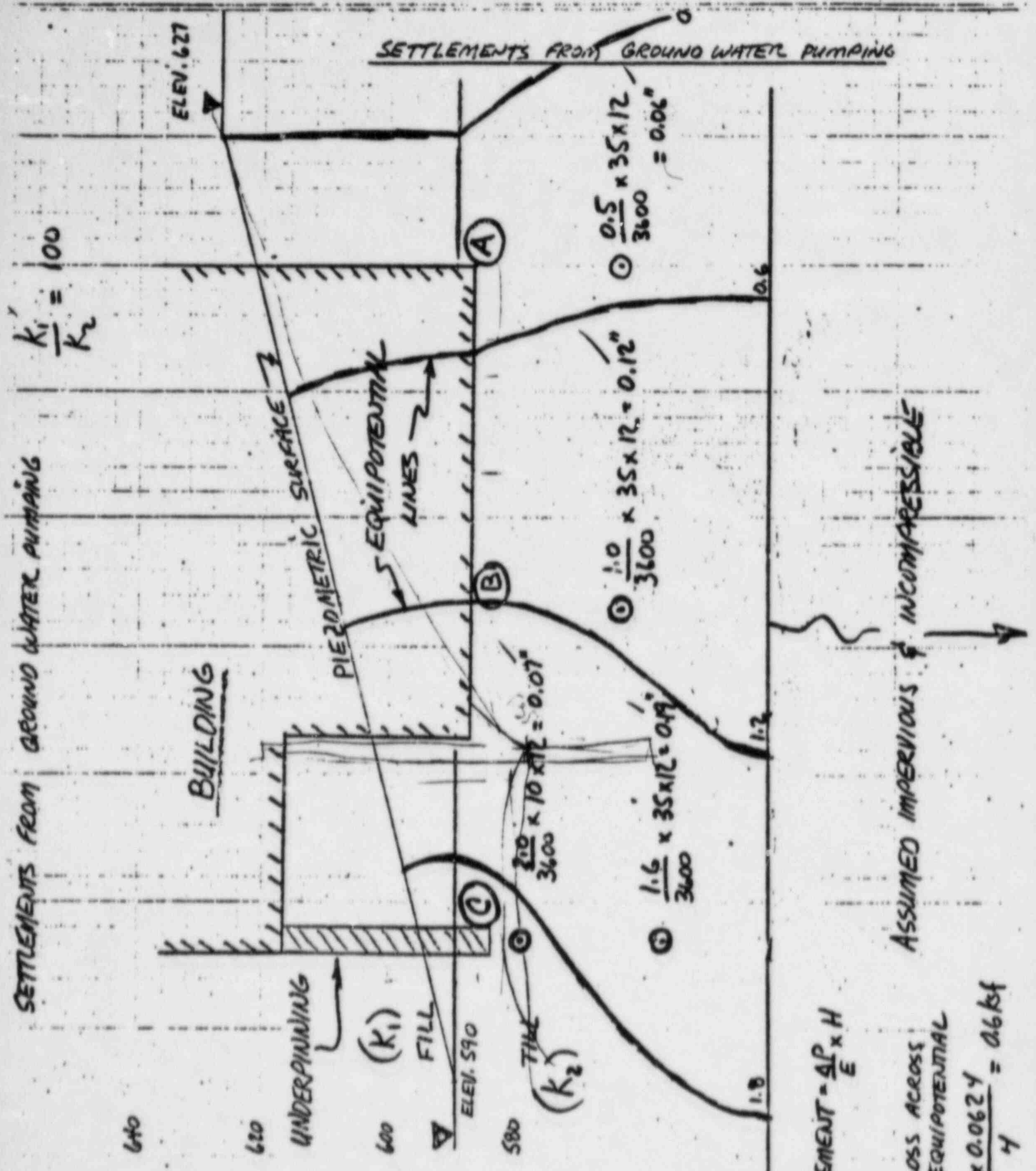

SUB 2
7220-C100-3-2 P28

Settlements from Groundwater Pumping

Location	Settlement
(A) Lake side of structure	0.06"
(B) Center of structure	0.1"
(C) At underpinning pier	0.26"

These values are derived on Sheet 14. The increase in effective stress in glacial till beneath SWPS is taken as the decrease in pore water pressures in a layer of till approximately 50 feet thick beneath the foundations.

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File 5345
Made By CHF Date 8-25-81
Checked By JPG Date 8-25-81

FORM 3

ESTIMATE SETTLEMENTS FROM WATER STORAGE LOAD

EQUIVALENT ADDITIONAL LOAD ≈ 2ksf (average over building area bearing directly on till)

ELASTIC MODULUS OF SOIL

LAYER	ELEV.	THICKNESS	E
A	585-582.5	2.5'	2,400 ksf
B	582.5-562	20.5'	3,600
C	562-543	19'	6,400
D	543-503	40'	6,400
E	503-363	140'	6,400

These E values now become significant because of the large size of the loaded area. Therefore they are increased to reflect observed settlements of containment structures

AREA OF STRUCTURE = 74' x 90'

REVISED
E VALUES TAKEN FROM RESPONSE TO QUESTION NO. 39

7220-C100-3-2 P28
SUB 2

① TRY ELASTIC SOLN.

REF. WHITMAN & RICHART, 1967, "DESIGN PROCEDURES for Dynamically loaded foundations", J. SMFE, ASCE, VOL 93, NO. SM6, pp 169-193.

$$\rho_z = \frac{P(1-\nu^2)}{\beta_z\sqrt{BL}\,E}$$

$$\rho_z = \frac{P(1-\nu^2)}{\beta_z\sqrt{BL}\,E} \quad \dots (7.17)$$

where P = total vertical load
B, L = rectangle dimensions
β_z = factor dependent on L/B and plotted in Fig.7.8.

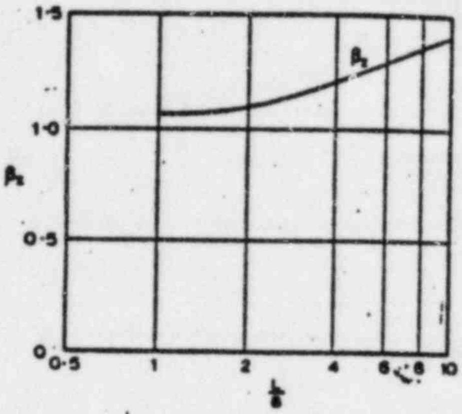

FIG.7.8 Coefficient β_z for rigid rectangle (Whitman and Richart, 1967).

$P = 2\text{ ksf} \times 74' \times 90' = 13320^{k}$
$\nu = 0.33$

$L/B = 1.22 \rightarrow \beta_z = 1.1$

$$\rho_z = \frac{13320 \times (1-0.33)}{1.1\sqrt{74\times 90}\times E} = \frac{132}{E}$$

$E = 4{,}800 \rightarrow \rho_z = 0.03' = \underline{\underline{0.3''}}$

$E = 6{,}400 \rightarrow \rho_z = 0.02' = \underline{\underline{0.2''}}$

Note: later information indicates that 2 ksf water reservoir loading is too high, possibly double the actual load, so that these values are too high by possibly 100%

Referring to DM-7 procedure on sheet Nos 1 & 2:

$C_s = 0.88$ — Assume no embedment

$$\delta = \frac{(1-\nu^2)\,q\,B\,C_s\sqrt{L/B}}{E_s} = \frac{(1-\overline{0.33}^2)\times 2\times 74\times 0.88\times\sqrt{1.22}}{E_s}\times 12 \text{ (in.)}$$

$\delta = \frac{1538}{E_s}$

E_s	δ (in)
4800	0.3"
6400	0.2"

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FOR Service Water Building

MADE BY CHF DATE 8-25-81
CHECKED BY JPG DATE 8-25-81

FORM 3

② ESTIMATE SETTLEMENTS FROM STRESSES UNDER A RECTANGULAR LOAD

REF. FADUM, R.E, 1948, "INFLUENCE VALUES FOR ESTIMATING STRESSES IN ELASTIC FOUNDATIONS", PROC., 2ND ICSMFE.

FOR RECTANGLE

$$\rho_{RIGID} \approx \frac{1}{3}\left\{2\rho_{CENTER} + \rho_{CORNER}\right\}_{FLEXIBLE}$$

REF. FOX, 1948, "THE MEAN ELASTIC SETTLEMENT OF A UNIFORMLY LOADED AREA AT A DEPTH BELOW THE GROUND SURFACE", PROC., 2ND ICSMFE, VOL. 1, Pg 129

LAYER	DEPTH TO MIDLAYER	$\frac{X}{Z} = \frac{37}{Z} = m$	$\frac{Y}{Z} = \frac{45}{Z} = m$	I	4I	STRESS @ MIDLAYER $= 4I \times Z$	SETTLEMENT $= \frac{\sigma}{E} \times H \times 12$
A	1.3'	28	35	0.25	1	2.0	0.03
B	12.8'	2.9	3.5	0.245	0.98	2.0	0.14
C	32.5'	1.1	1.4	0.21	0.84	1.7	0.06
D	62'	0.60	0.73	0.125	0.50	1.0	0.08
E	152'	0.24	0.30	0.04	0.16	0.3	0.08

@ CENTER Σ 0.39"

LAYER	DEPTH	$m = \frac{74}{Z}$	$n = \frac{90}{Z}$	I	STRESS	SETTLEMENT
A	1.3	57	69	0.25	0.5	0.01
B	12.8	6	7	0.25	0.5	0.03
C	32.5	2.3	2.8	0.24	0.5	0.02
D	62	1.2	1.5	0.21	0.4	0.03
E	152	0.5	0.6	0.1	0.2	0.05

Σ 0.14

$$\rho \approx \frac{1}{3}\left[2 \times 0.39 + 0.14\right] = 0.3"$$

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CONSULTING ENGINEERS

FORM 8

FOR MIDLAND

FILE 5345
MADE BY SRS DATE 3/27/81
CHECKED BY LH DATE 8/19/81

SUMMARY TABULATION FOR TOTAL DEFORMATION

SEE PLATE 5

TIME (DAYS)	0	1	2	3	4	5	10	90	100	1000	10,000
SETTLEMENT		.03	.07	.18	.29	.32	.34	.35	.39	.44	.49
SHRINKAGE		.01	.01	.02	.025	.03	.04	.19	.19	.20	.2
CREEP		.01	.011	.012	.0125	.013	.016	.028	.029	.031	.031
SUMMATION		.05	.09	.21	.33	.36	.40	.57	.61	.67	.72

TOTAL-DEFLECTION AFTER 100 DAYS ≈ .61"

TOTAL MAX DEFLECTION 10,000 DAYS = .72

Δ = .11" = 1/8"

7220-C100-3-2 P31
SUB 2

VALUES: FINAL SHRINKAGE/INITIAL ELASTIC DEFORMATION
FINAL CREEP

TOTAL LOAD ON CONCRETE: 4'x5' PIER (NO BELL)

1) FINAL JACK LOAD $\frac{3500}{86} = 40.7^{K}/LF \times \frac{1}{4} = 10.2^{K}/SF$

2) AVG CONC. WT : $\frac{150}{1000} \times 32/2 =$ $2.4^{K}/SF$

3) AVG FRICTION $\approx 6.2 \times (\frac{1}{2}) = 3.1^{K}/LF \times \frac{1}{4} = .8^{K}/SF$

13.4 K/SF

$13.4 \frac{K}{SF} \times 1000 \times \frac{1}{144} = 93.06$ psi ⇒ SAY $\boxed{100 \text{ psi}}$

SHRINKAGE: FINAL SHRINKAGE S_f $\boxed{\text{SEE PLATES 3 \& 4}}$

USING 310# of H_2O/CY of CONCRETE
(THIS WOULD BE WATER USED FOR $f'_c = 4000$) <SEE PLATES>

$S_f = .00052$ in/in DRYING SHRINKAGE

TOTAL SHRINKAGE $.00052 \times 32' \times 12''/_1 = .20''$

CREEP: TOTAL (FINAL) CREEP
$\boxed{\text{SEE PLATE 2}}$

SPECIFIC CREEP $.8 \times 10^{-6}$ /psi
FOR $f'_c = 4000$

$\epsilon_{S_{CREEP}} = 100 \text{ psi} \times .8 \times 10^{-6}$
$= 80 \times 10^{-6}$
$= .00008$

$C_f = .00008 \times 32 \times 12$
$C_f = .0307''$ $\sim \frac{1}{32}''$ TOTAL (FINAL) CREEP

ϵ_{INST}: ELASTIC DEFLECTION
$\cong$ TO $\frac{1}{3}$ FINAL VALUE

INSTANTANEOUS:
$.267 \times 10^{-6}$ /psi $\times 100 = 26.7 \times 10^{-6} = .0000267$

$C_{INST} = .0000267 \times 32 \times 12 = .010''$ INITIAL

7220-C100-3-2 P32

TOTAL $\Delta = S_f + C_f = .20 + .03 = .23''$ SHRINKAGE + CREEP

SUB 2

FORM 8

SHRINAKE: AT PIER — SEE PLATE 4

HT OF PIER = 32' × 12"/' = 384"

TIME (DAYS)	DECREASE IN HT. in/in ×10^{-3}	HT. OF PIER INCHES	SHRINKAGE INCHES
10	.12	384"	.046"
20	.20		.077"
40	.32		.123"
80	.4		.154"
120	.49		.188
140	.50		.192
160	.51	384"	.196"
200	.52	384	.2"

ULTIMATE SHRINKAGE @ 200 DAYS = .2"

SHRINKAGE @ 100 DAYS = .19"

Δ = .01" ±

REF: PRESTRESSED CONC. INSTITUTE
MANUAL ON DESIGN
Pg 86

FORM 2

CREEP AT PIER @ 100 psi

SEE PLATE 2

TIME (DAYS)	UNIT CREEP in/in/psi $\times 10^{-6}$	f_c psi	HT. OF PIER in.	CREEP in.
0	0	100	384	0
0 +	.267			.01
10	.3			.012
20	.39			.015
30	.46			.018
40	.54			.021
50	.59			.023
60	:			:
70				
80	.71			.027
90	.74			.028
100	.76	100	384	.029
:				:
FINAL	.8	100	384	.0307" ⇒ .3"

will be repeated until a distribution of jacking loads that maintains building stresses within allowable limits is achieved. The computed adjusted jacking loads will be the final jacking loads used for construction. It is not anticipated that adjustments will amount to more than 20 percent, and the design of the underpinning structure is more than ample to accommodate increases of this magnitude.

5.3 BEARING PRESSURES

5.3.1 Preliminary Calculated Bearing Pressures

The maximum bearing pressure under the underpinning wall produced by final jacking load alone amounts to 6.8 ksf at the north underpinning wall. The analysis described in Section 8.2 below indicates that the safety factors against various load combinations which incorporate the bearing pressure for jacking loads exceed by large margins those safety factors committed to for foundation conditions in the PSAR. A summary of those results is as follows:

52 ksf

Loading Conditions	Safety Factor for Ultimate Bearing Capacity of 48 ksf
Temporary peak loading during jacking incl. maximum downdrag and no seismic load 6.8 + 0.5 + 4.7 = 12 ksf	4.2
Long-term sustained loading, including eventual downdrag and no seismic load 6.8 + 0.5 + 2.0 = 9.3 ksf	5.4
Long-term sustained loading, including eventual downdrag, plus seismic load 9.3 + 6.4 = 15.7 ksf	3.2

Revise

This table should conform to Table SWP-2

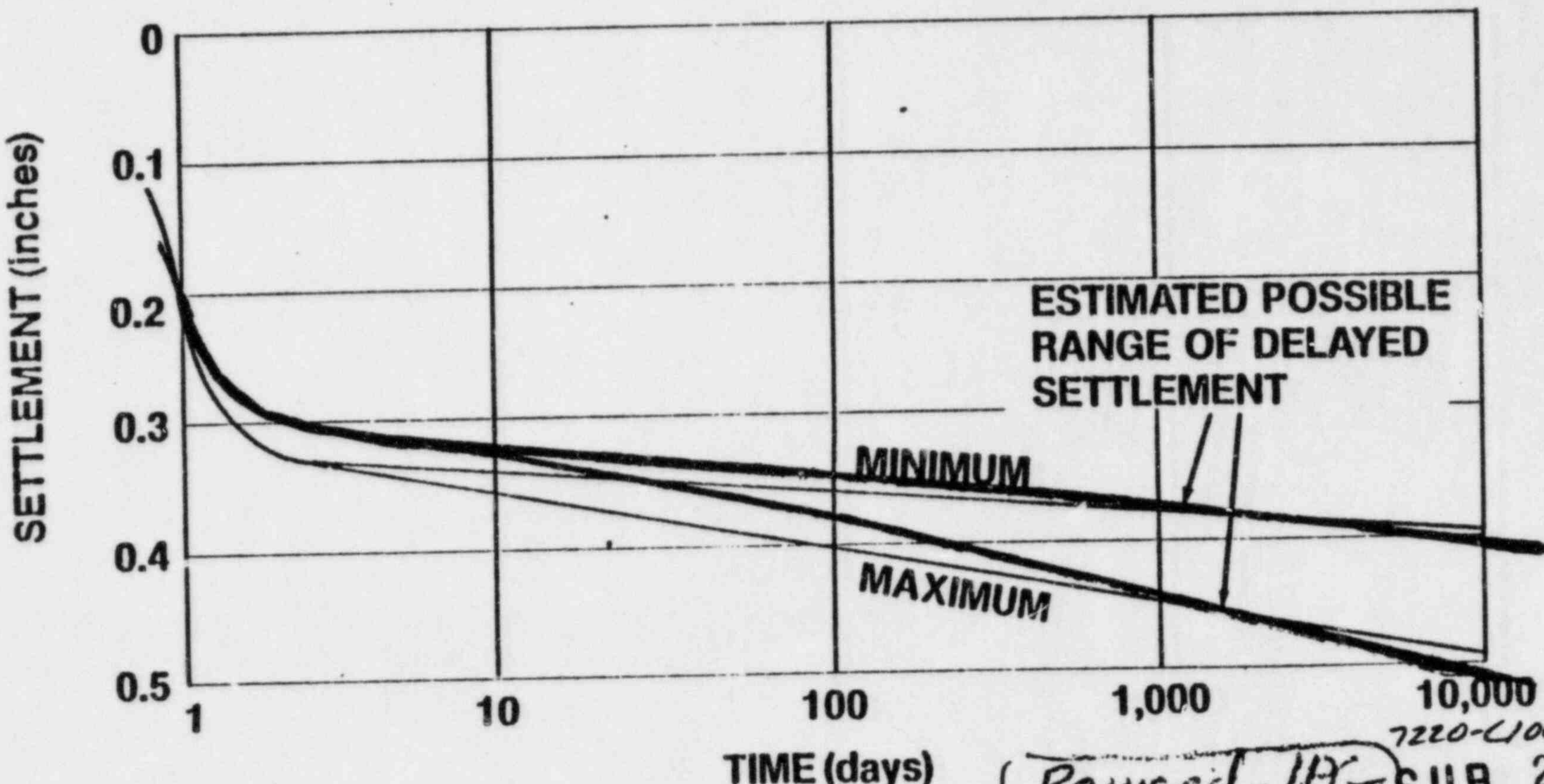
7220-C100-3-2 P35

SUB 2

SERVICE WATER PUMP STRUCTURE

ESTIMATED TOP OF PIER DEFLECTION DUE TO CONSOLIDATION OF SOIL VS TIME

(Time Is Measured from Start of Jacking)



7220-C100-3-2

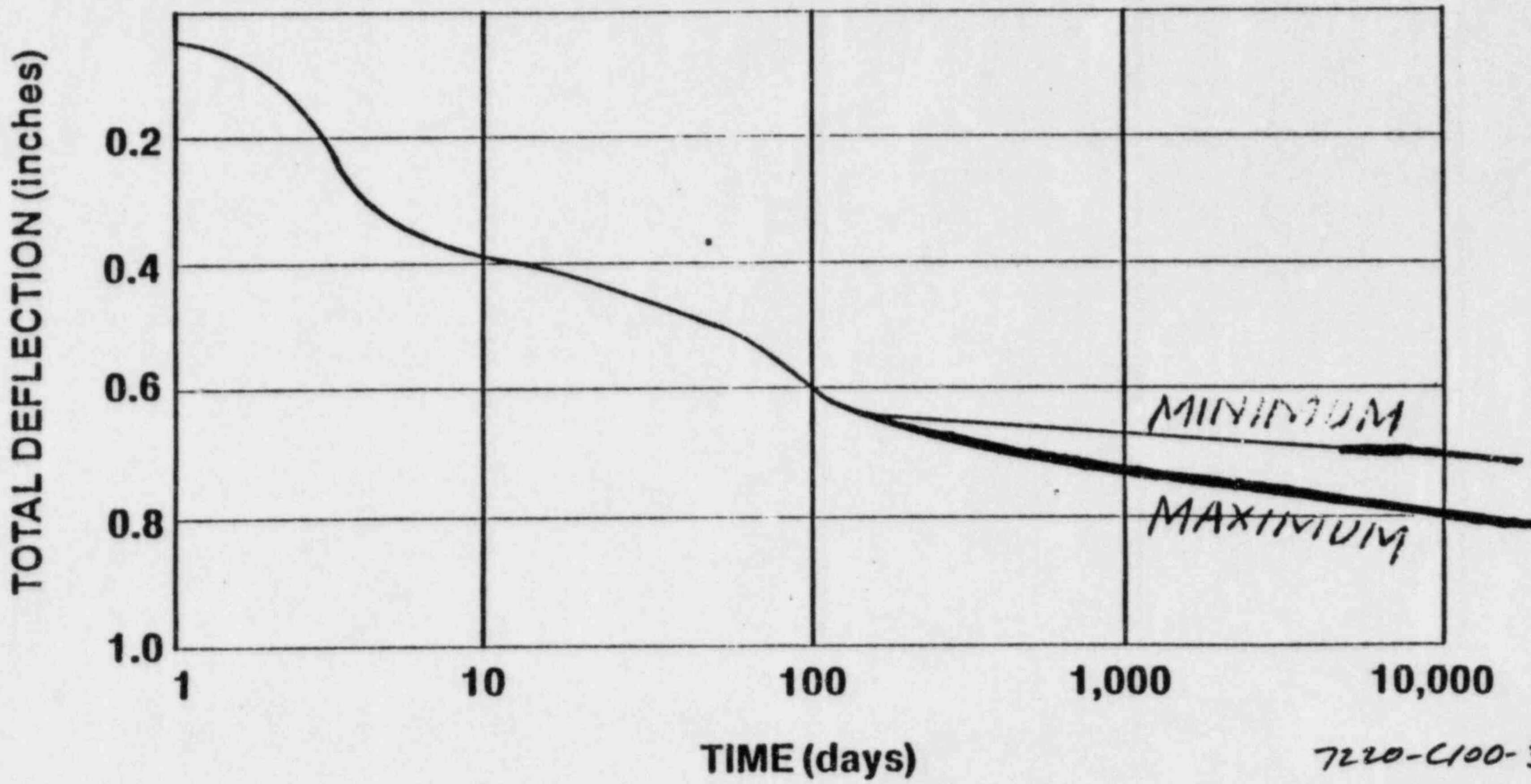
Revised JPG 2-1-82

SUB 2

Q-1584-14 P36

FIG SWP-24

SERVICE WATER PUMP STRUCTURE
ESTIMATED TOP OF PIER DEFLECTION DUE TO TOTAL DEFORMATION VS TIME
(Based on Maximum Consolidation of Soil vs Time)



7220-C100-3-2

G-1584-12

Revised JPG
2-1-82

FIG SWP-25 P37

SUB 2

FORM 2

MIDLAND PLANT UNITS 1 & 2

CONSUMER POWER COMPANY

SERVICE WATER INTAKE STRUCTURE

DESIGN CALCULATIONS

STUDY OF UNDERPINNING & SETTLEMENT ANALYSIS

DESIGNERS & CHECKERS	INITIALS
WALTER WALCHUK	WW
CARSTEN H. FLOESS	CHF
STUART K. SOKOLOFF	SKS
PETER EDINGER	PE
LEN Y. HOM	LH
JAMES P. GOULD	JPG

SERVICE WATER INTAKE STRUCTURE
STUDY OF UNDERPINNING

STATEMENT OF PROBLEM:

PORTION OF THE SERVICE WATER INTAKE STRUCTURE IS SUPPORTED ON COMPACTED FILL. TO PREVENT POTENTIAL FUTURE SETTLEMENTS, DECISION WAS MADE TO UNDERPIN THIS PORTION OF THE STRUCTURE TO THE UNDERLYING TILL LEVEL.

DESIGN CRITERIA

THE PROPOSED UNDERPINNING IS TO BE DESIGNED TO TRANSMIT ALL BUILDING AND SEISMIC LOAD TO THE FIRM SUBGRADE LEVEL.

SOURCES OF DESIGN CRITERIA

BECHTEL INFORMATION DRAWINGS SK-C-748, REV A AND SK-C-749, REV. A OF MARCH 6, 1981, PLUS BECHTEL'S STRUCTURAL & PIPING DESIGN DRAWING AND AVAILABLE SOIL BEARING DATA AS OUTLINED IN THE "CONSUMERS POWER COMPANY" FINAL SAFETY ANALYSIS REPORT, VOLUME 4.

ASSUMPTIONS SUB 2

AS LISTED ON INDIVIDUAL PAGES OF CALCULATIONS.

DISTRIBUTION:
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RLandsman ACRS (16)
FRinaldi

APR 2 1982

Docket Nos: 50-329 OM, OL
and 50-330 OM, OL

Mr. J. W. Cook
Vice President
Consumers Power Company
1945 West Parnall Road
Jackson, Michigan 49201

Dear Mr. Cook:

Subject: Staff Concurrence for Installation and Operation of Construction Dewatering and Observation Wells for the Service Water Pump Structure

REFERENCES:
1) J. W. Cook letter of March 23, 1981, announcing bin wall concept
2) Structural Design Audit, April 20-24, 1981, Ann Arbor
3) J. W. Cook letter of August 26, 1981, with "Technical Report on Underpinning the SWPS"
4) Meeting of September 17, 1981, on SWPS underpinning
5) J. W. Cook letter of November 6, 1981, providing information requested during September 17, 1981, meeting
6) J. W. Cook letter of November 6, 1981, with report "Test Results, SWPS Soil Boring and Testing Program"
7) J. W. Cook letter of March 2, 1982, with report "Evaluation of Cracking in SWPS at Midland Plant"
8) Meeting of February 23-26, 1982
9) J. W. Cock letter of March 2, 1982, with report "SWPS Three-Dimensional, Finite-Element Models"

By an audit meeting on March 16-19, 1982, and by several referenced reports and meetings, you have described the remedial underpinning planned for the Service Water Pump Structure (SWPS) for Midland Plant, Units 1 and 2. Preparations for this underpinning activity include temporary dewatering of the immediate area by approximately 65 construction wells to be located inside the SWPS, inside the adjacent Circulating Water Intake Structure, and the remaining perimeter outside the SWPS. Staff concurrence to proceed with this dewatering was requested by Mr. J. Mooney of your Company on March 17, 1982.

During the audit meeting on March 17, 1982, the staff was provided copies of the subcontractor's plan (Enclosure 1) and the preliminary procedure (Enclosure 2) for construction dewatering. Several changes to the procedures were discussed including:

(1) The method of well placement will be changed to the method previously accepted by the staff for the interceptor and area permanent dewatering wells.

~~8204210024~~ 18pp.

OFFICE							
SURNAME							
DATE							

(2) The depths of the dewatering wells should be increased so as to maintain the water table below the bottom of the underpinning pier foundations.

(3) Consumers Power Company is considering an optional approach for the excavation drift which would relocate the drift from beneath the structure to the exterior edges of the structure, where those exterior walls are accessible (i.e., along the northwest and northeast walls). Location of the construction wells would be positioned to accommodate this option.

These and other matters regarding the construction dewatering plans were further discussed during a telephone discussion on March 26, 1982 (Enclosure 3). Pages 4 and 5 of Enclosure 3 identify several changes which were agreed to as a result of staff recommendations. We have discussed Q-listed aspects of this dewatering with our Region III Office and have included the following items from Enclosure 3 to be of particular interest in this respect:

1. That the depths for piezometers and filter sand identified by paragraph 3a be achieved;

2. That the minimum two foot depth between the upper phreatic surface and the bottom of any open underpinning excavation, as discussed by paragraph 3c, be maintained; and

3. That monitoring for the loss of fine soil particles be performed consistent with the compromise position discussed in paragraph 3e.

On the basis of our review of the information provided, and the intended changes identified which Consumers has committed to making, the staff agrees with your plan to proceed with construction dewatering for the Service Water Pump Structure area.

This confirms the verbal concurrence by our Project Manager to Mr. J. Mooney on March 26, 1982.

Sincerely,

Original signed by
Robert L. Tedesco

Robert L. Tedesco, Assistant Director
for Licensing
Division of Licensing

Enclosures:
As stated

cc: See next page

							AD:L/DL
							RTedesco
							4/ / /82
OFFICE	DL:LB #4	LA:DL:LB #4	HGEB	HGEB	AD:CSE	OELD	LB:#4/DL
SURNAME	DHood/hmc DSIT	MDuncan	JKane JDK	GLear	JKnight	~~WPaton~~ BLUME	EAdensam
DATE	4/1/82	4/ /82	4/1/82	4/1/82	4/1/82	4/1/82	4/1/82

MIDLAND

Mr. J. W. Cook
Vice President
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Ms. Barbara Stamiris
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Mr. Steve Gadler
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Geotechnical Engineers, Inc.
ATTN: Dr. Steve J. Poulos
1017 Main Street
Winchester, Massachusetts 01890

Service Water Pump Structure - Dewatering Procedure

The following procedure is as submitted by the subcontractor and is in the process of being reviewed. Revisions will be required if the access drifts are relocated outside.

4.0 OVERALL DESCRIPTION OF THE SYSTEM

4.1 Approximately 42 dewatering wells will be installed at 6± feet on center along the north and east sides of the SWPS, about 9 feet outside the proposed access excavation for the underpinning. These wells will be from existing grade, elevation 634± and will consist of 6" I.D. PVC well screens encased in select sand filters for their full depths.

4.2 Approximately 18 dewatering wells will be installed from within the SWPS, north of the northermost wall extending down to elevation 592. These wells will be installed from elevation 620. Approximately 11 dewatering wells will be installed from within the Circulating Water Intake Structure (CWIS) west of the easternmost wall extending down to elevation 610. These wells will be installed from elevation 610. All wells located within structures will be spaced at an average 4± feet on center and will consist of a 3" I.D. galvanized low carbon steel screen.

4.3 All dewatering wells will be located to miss existing utilities, pipe lines, etc. All locations will be submitted to Bechtel on shop drawings for approval. All wells will extend to the bottom of the deepest sand stratum or to a minimum elevation of 585 where clay till is encountered above that minimum. Prior to the installation of drilling of any well, a drill permit (see Attachment A) will be obtained from Bechtel.

4.4 Two inch (2") eductor pumping units will be used to pump the wells. The eductor pumping units will be of sufficient size and number to lower the water in each well to the bottom of the well. The eductor pressure and return headers, 8" in diameter, will be located to suit the site conditions and will connect all eductor units to a control pump station. The pump station will have an electric operating pump and 100% stand-by diesel powered pump capable of operating the eductor units. The discharge from the dewatering system will be piped to the pond. The volume of water pumped will be recorded by a meter. The dewatering system will be installed and maintained in operating condition for the duration of the subcontract although pumping may be discontinued when not required.

4.5 Piezometers (observation wells) will be installed at the approximate location shown on the subcontract drawings.

4.6 Approximate locations of major components of the dewatering system will be shown on the dewatering shop drawing which will be submitted to Bechtel for approval. Installation will not proceed until said approv is granted.

5.0 EQUIPMENT AND MATERIALS

5.1 Dewatering well screens will consist of 6" I.D. slotted PVC pipe for wells outside the structures and 3" I.D. continuous slot galvanized low carbon steel as manufactured by Loughney Dewatering Inc. Slot size will be 0.018 inches for the 6" I.D. PVC screen and 0.008 inches for the 3" I.D. galvanized steel screen.

5.2 Two inch eductor pumping units will have an approximate capacity of 7 gallons per minute (GPM).

5.3 The electric operating pump will have a motor rated at 150 horsepower (HP) and will be capable of pumping 800 GPM at an approximate pressure of 125 pounds per square inch (PSI).

5.4 The stand-by diesel power pump will also have a motor rated at 150 HP and be capable of pumping 800 GPM at approximately 125 PSI.

5.5 The meter for measuring discharge volume will be a "Badger" flowmeter sized according to the discharge volume which develops.

5.6 Observation well screens will be 1¼ inch I.D. by 3 feet long slotted PVC. The riser pipe will be 1¼ inch I.D. PVC.

5.7 The select filter sand will have approximately the following gradation:

Sieve Size	Percent Passing
4	100
10	90-100
16	75-100
20	60- 98
30	25- 40
40	3-15
50	0- 2

5.8 The filter for dewatering wells inside the structures and for the observations wells will consist of Ottawa Flint Shot (OFS) sand.

5.9 Drilling mud will be mixed using "Revert" as manufactured by U.O.P. Johnson Company or an equivalent organic drilling fluid.

6.0 INSTALLATION PROCEDURE

6.1 Devatering wells outside the structre:

6.1.2 Hydraulic Rotary Drilling Method - A rotary drilling rig capable of drilling a 14" diameter hole to an 80 foot depth, such as Failing 1500, will be used to drill the holes for the dewatering wells. A 6' x 6' x 5' pit will be dug and using a venturi type mixer, Revert will be mixed in the pit and then pumped through the top of the rotating drill stem and out the bottom of the 14" diamter bit. The Revert and the soil cuttings from the drilled hole will be returned up around the outside of the drill pipe and conveyed to the pit through a ditch, where most of the cuttings will be settled out in the bottom of the ditch and the pit. The procedure will be repeated until the hole has been drilled to the desired depth. After which a 2" pipe will be inserted to the bottom of the hole and clean water will be added and the cuttings remaining in the Revert fluid will be flushed out of the hole and disposed of in a controlled manner. The drills rods and bit will be removed. A 6" diameter PVC well screen and riser will be installed in the hole and centered. The well screen will extend from the bottom of the hole to within 7 to 10 feet of the ground surface. The last 7 to 10 feet will be 6" PVC riser. The annulus between the 6" well screen and the hole will be filled with a select filter sand. The filter sand will be placed by shoeveling it into the annulus and the 2" water line gradually removed as the filter is placed. The procedure will be repeated for each dewatering well.

6.2 Dewatering Wells Inside the Structures - The eductor wells within the SWPS and the CWPS will be installed as follows: Where the groundwater level is above elevation 620 and 610 provisions will be made to balance the hydrostatic head at each proposed eductor well during its installation with a 6" casing sealed in the concrete, and the casing will extend above the groundwater level. A hole of sufficient diamter to accommodate a 4" I.D. casing with a 4" I.D. open drive shoe on the bottom will be drilled through the concrete. The 4" I.D. casing and open drive shoe will be driven to the bottom of the deepest sand stratum or to elevation 585 where the clay till is encountered above elevation 585. The casing will be cleaned out with clean water as it is being advanced. At no time will a slurry, drilling mud, Revert or similar additives be used during the installation of the 4" I.D. casing. At all times the water level in the 4" I.D. casing will be above the prevailing groundwater level. A 3" I.D. well screen and riser will be installed inside of the 4" casing and then the casing will be removed. The well screen will extend from the bottom of the hole to within 2 feet of the bottom of the concrete floors. The section of the well above the screen will be 3" galvanized steel pipe and it will be sealed in the concrete. Any void space remaining between the well screen and the soil will be filled with OFS sand. The procedure will be repeated for each well.

6.3 Piezometers - Piezometers shall be installed at an elevation no lower than 2'-0" above the original undisturbed natural material, as determined by Bechtel's resident Geologist. A chart will be prepared that will show the location of each piezometer; its number; its top elevation, its tip elevation, the depth to water, and the elevation of the water level. This chart will be transmitted to Bechtel within one week of each piezometer's respective completion date. Piezometers 1 and 2 shall be installed at ground elevation prior to the installation of the access shafts. Piezometers will be installed by two methods based on their respective location.

6.3.1 - Piezometers outside the structures will be installed in the following manner. At each location a hole of sufficient diameter to accommodate a 4" I.D. pipe will be rotary drilled to within 6 feet of the proposed tip of the observation well using Revert as the drilling fluid. A minimum of 4" I.D. casing will be set in the drilled hole to the bottom of the hole. The Revert will be flushed out of the 4" casing with clean water. Using the clean water the hole will be drilled to the tip elevation; the hole flushed with water; one foot of select filter sand placed at the bottom; an observation well screen and riser placed in the hole; the hole filled with sand for a depth of 5 feet; the casing raised 2 feet; the hole filled with one foot of bentonite seal; the casing raised 2 feet and the hole filled with sand in 2 foot increments. This procedure is repeated until the casing completely is removed. There is only one bentonite seal.

6.3.2 Piezometers located within the structures will be installed in the following manner. Provisions will be made to balance any hydrostatic head that is above the top of the concrete floors elevation 610 and 620 at each observation well location with an exterior casing sealed in the concrete. At each location a hole of sufficient diameter to accommodate a 4" I.D. pipe will be drilled through the concrete. Below the concrete the hole will be drilled using Revert to within 6 feet of the proposed tip of the observation well. A minimum of 4" I.D. casing will be set in the drilled hole and extended from the working level to the bottom of the hole. The Revert will be flushed out of the 4" casing with clean water. Using clean water the hole will be drilled to the tip elevation; the hole flushed with water; one foot of select filter sand placed at the bottom; observation well screen and riser placed in the hole; the hole filled with Ottawa Flint Shot sand for a depth of 5 feet; the casing raised 2 feet; the hole filled with one foot of bentonite se casing raised 2 feet; the hole filled with one foot of bentonite se the casing realsed 2 feet and the hole filled with sand in 2' incre This procedure repeated to the top of the concrete floors elevatio 620 or 610. There is only one bentonite seal.

At all times during the installation the water level in the 4" cas the exterior casing and the hole will be maintained at least 7 fee above the prevailing groundwater level.

Where the groundwater level is above the top of the concrete floor elevation 620 or 610 the 4" casing will be removed and the exteric casing and the observation well riser will be left at or above the groundwater level. Where the groundwater level is below the elev of the concrete floor the observation well riser will be cut off slightly above the top of the concrete floor and the riser sealed the concrete floor.

6.4 After completion of the dewatering and observation well instal records for these wells as required by ACT 218 P.A. 1972, which is amendment to ACT 294 P.A. 1965, Ground Water Quality Control Act w be prepared. The dewatering well record form shall either be comp for every well or a composite record made for several wells. The composite record may be used where the subsurface conditions are similar, the surface relief relatively level, and the static water level at a constant depth. Depending on variations in subsurface conditions, one or several composites may be necessary. Drillers l. the observation well holes and construction as built details will be prepa

Copies of these records shall be submitted to Bechtel.

7. OPERATION AND MAINTENANCE

7.1 Development - After each well is installed it will be developed and tested in accordance with the established procedures and upon approval activated. During the well development tests SWP and Loughney personnel will aid Bechtel in any manner required.

All observation wells will be tested by either a pumping test or a falling head test to ascertain whether they are functioning properly.

During initial connection of the eductor pumps to the header system suitable petcocks, bushings, and nipples will be installed at each dewatering well for obtaining water quality samples.

7.2 The dewatering operation shall be controlled so the amount of soil particles in the discharge water is limited to 10 ppm. The water level in the observation wells and the volume of water being pumped and the operating pump pressure will be recorded once each day five days a week. The stand-by diesel pump will be started once a week. The component parts of the dewatering system will be checked routinely and adjustments made as required.

8.0 REMOVALS

8.1 Dewatering wells buried or left in place under or near the structures shall be sealed with grout after the dewatering operation is discontinued, in accordance with the most recent Michigan Wells Act and to the satisfaction of Bechtel. Approval of Bechtel shall be obtained prior to grouting wells.

8.2 All holes drilled in the SWPS and CWIS for use in dewatering shall be repaired using materials furnished by Bechtel and in accordance with Section 10.5 of Specification 7220-C-194(Q) Revision 1.

8.3 All piezometer holes shall be sealed with grout after the dewatering operation is discontinued.

9.0 REVISIONS

Should it develop that the conditions encountered during the installation or operation require deviations in the aobve stated procedure, the modifications deemed necessary will be submitted to the on-site geotechnical engineer for approval.

Attachment A

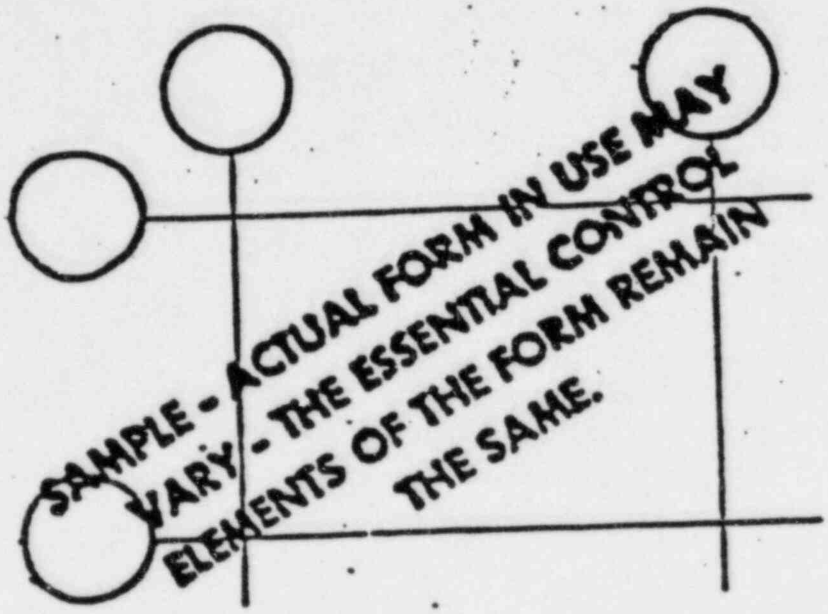
CONCRETE DRILLING PERMIT

Project 7220 Permit No.__________ Page ______ of ______

Prepared By:__________ Discipline:__________ Date__________

Unit__________ Bldg.__________ Area__________ Elev.__________

Location of Q-Listed blockwall or poured wall substituted for Q-Listed blockwall	Resident Engineer approval for attachment to Q-Listed blockwall or poured wall substituted for a Q-Listed blockwall. Appv'd ______ Not Appv'd ______ Date __________ By: __________



Notes: (1) SEE LATEST DESIGN DRAWING REVISION AND CHANGE ADDENDA PRIOR TO DRILLING.
(2) DO NOT CUT REBAR WITHOUT FIELD ENGINEERS APPROVAL. NO REBAR CUTTING IS ALLOWED IN Q-LISTED CONCRETE BLOCKWALLS OR POURED WALLS SUBSTITUTED FOR Q-LISTED CONCRETE BLOCKWALLS.

Specific Instruciton and Location Tolerance:__________

Note: If rebar encountered, notify __________.

If ground cable encountered, notify __________.

Before moving hanger, notify __________.

Loads per drawing C-2050: (list only for attachments to Q-Listed blockwalls or poured walls substituted for Q-Listed blockwalls)

Reference Drawings:__________

Approved By: (Not required for attachments to Q-listed blockwall or poured walls substituted for Q-listed block walls.)

Civil__________ Date__________ Piping__________ Date__________

Elect__________ Date__________ Instru__________ Date__________

Mech__________ Date__________

7220-C194-3-1

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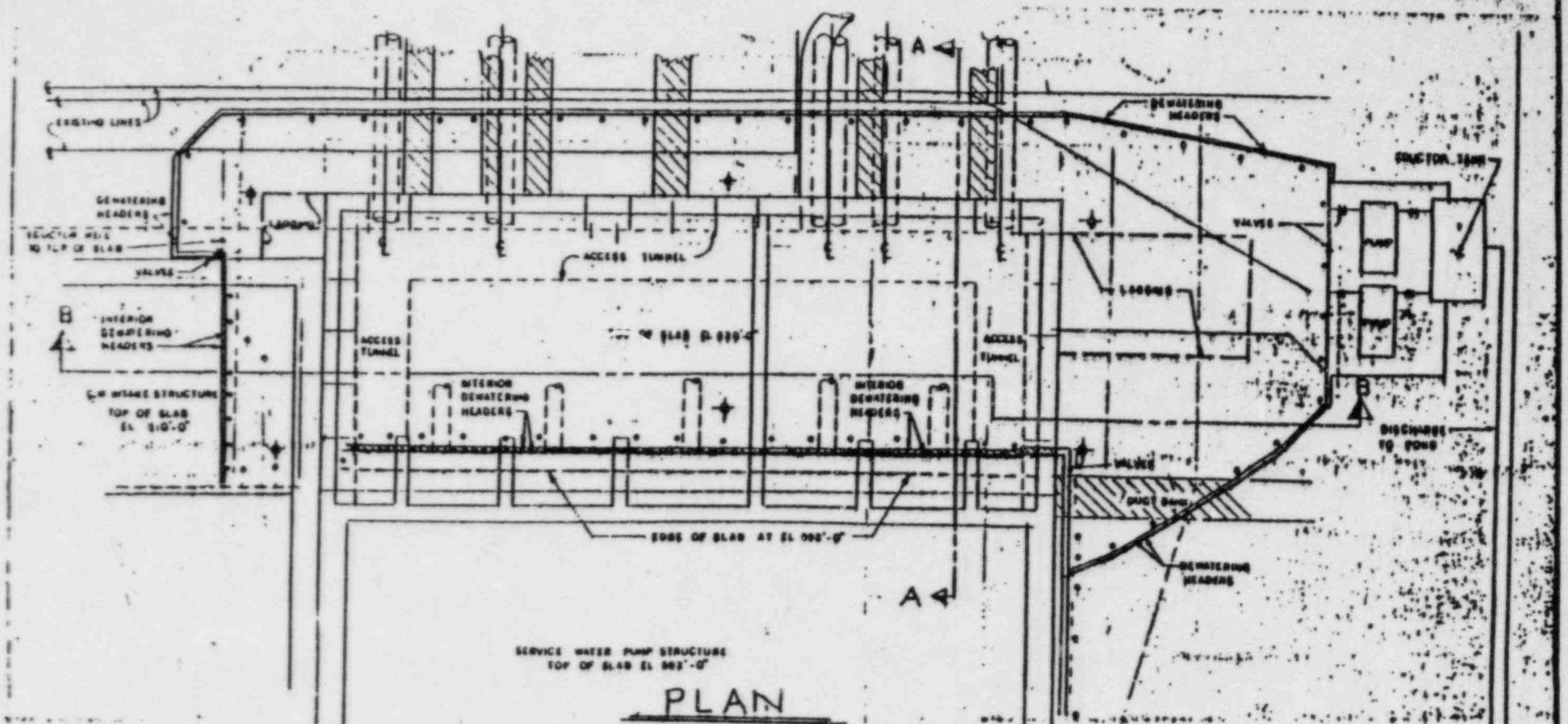
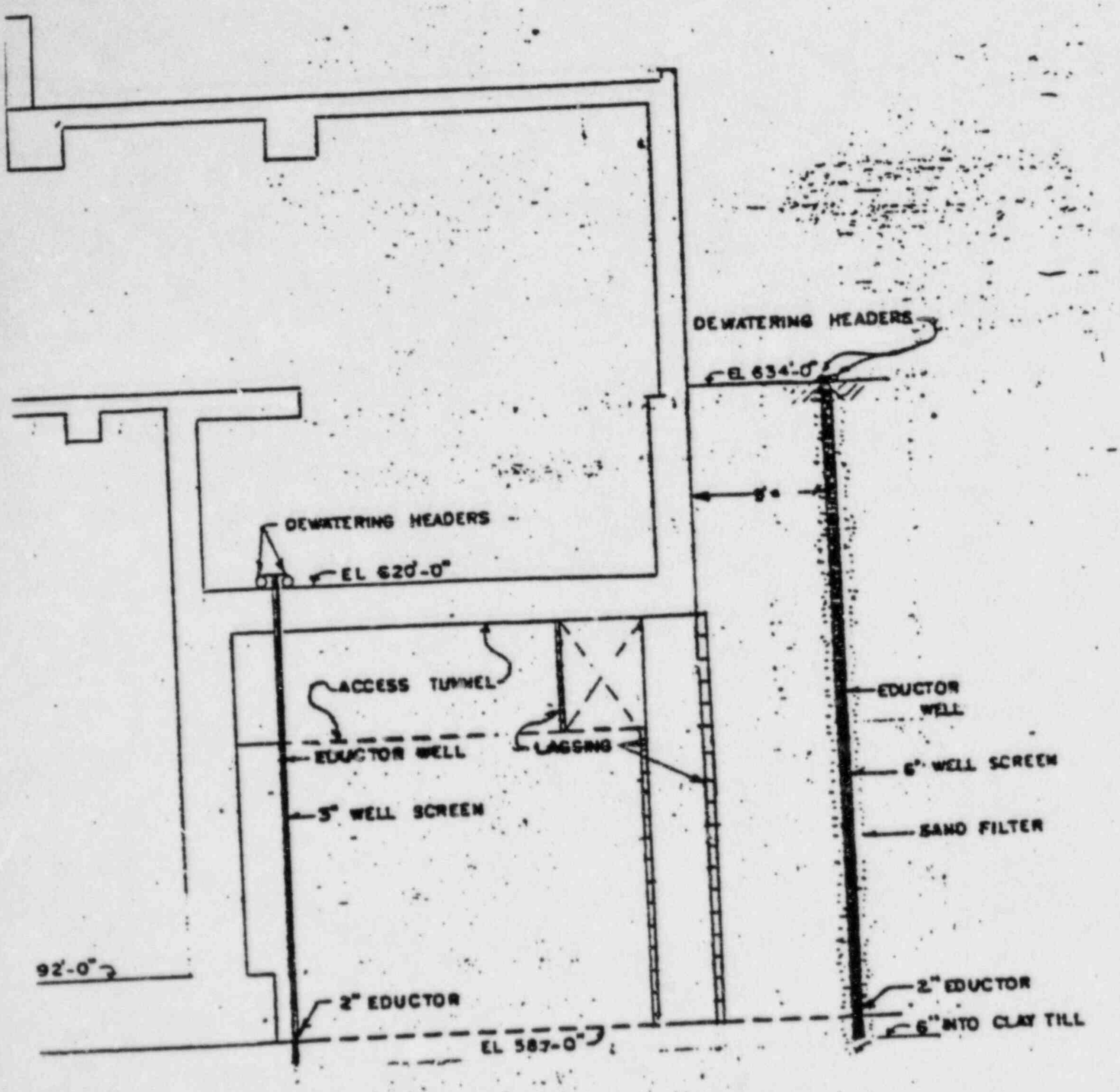


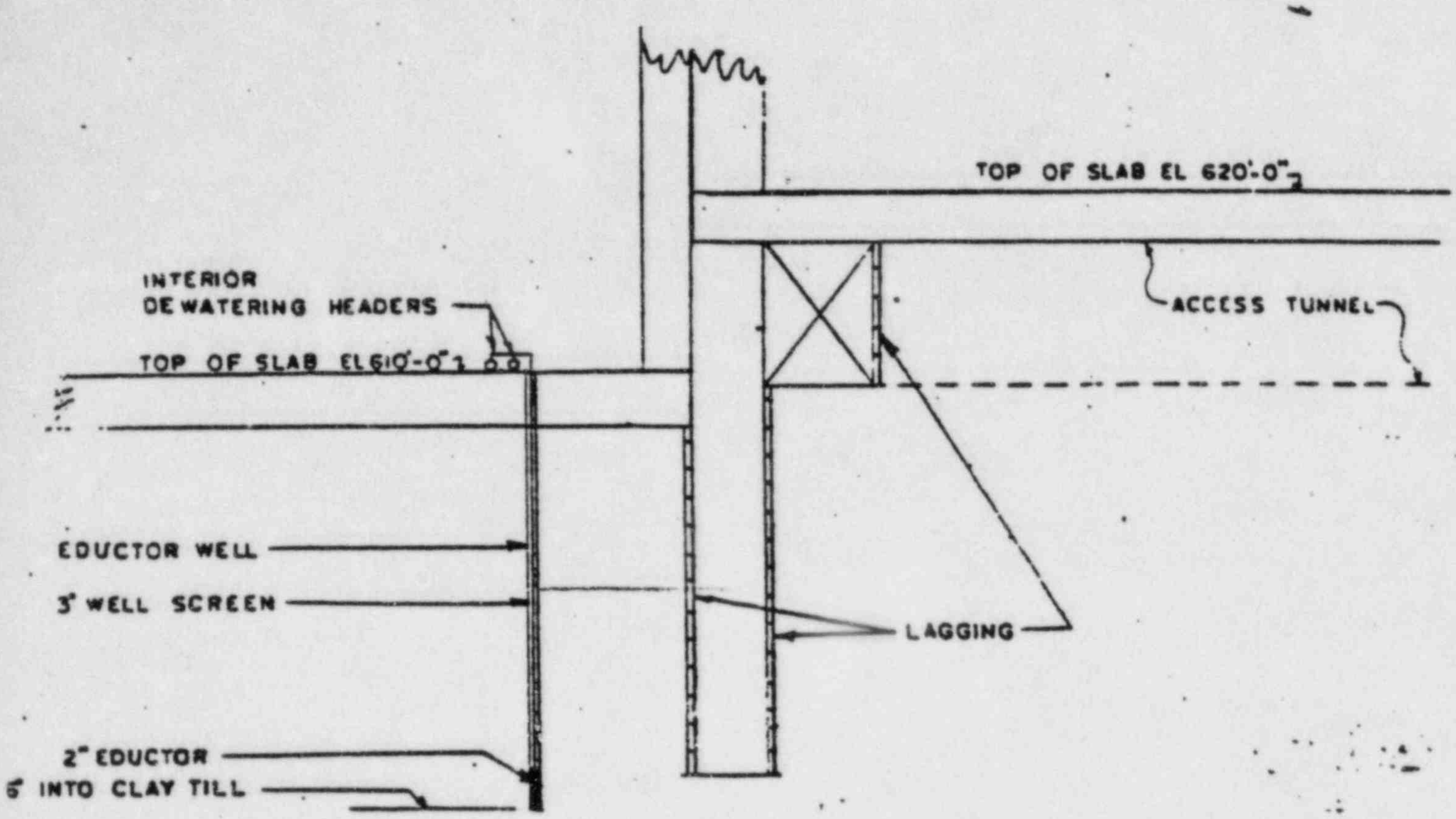
PLAN


DEWATERING HEADERS
EL 634'-0"
DEWATERING HEADERS
EL 620'-0"
ACCESS TUNNEL
EDUCTOR WELL
LAGGING
EDUCTOR WELL
5" WELL SCREEN
6" WELL SCREEN
SAND FILTER
92'-0"
2" EDUCTOR
2" EDUCTOR
6" INTO CLAY TILL
EL 587-0"

SECTION A-A

3/11/

Page



SECTION B-B

ENCLOSURE 3

RECORD OF TELEPHONE CONVERSATION

DATE: March 26, 1982 8:30 am to 11:30 am PROJECT: Midland

RECORDED BY: Joseph Kane CLIENT:

TALKED WITH:

CPC	Bechtel	COE	GEI	NRC
T. Thiruvengadam	J. Anderson	H. Singh	S. Poulos	J. Kane
K. Razdan	L. McKelvey			
R. Ramanujam	C. Russell			
	W. Paris			
	S. Afifi			
	N. Swanberg			

ROUTE TO: Information

G. Lear	S. Poulos
L. Heller	H. Singh
D. Hood	R. Landsman
F. Rinaldi	R. Gonzales

MAIN SUBJECT OF CALL: The purpose of this call was for the Staf' to respond to CPC on three review issues which had been discussed during the March 16-19, 1982 audit. During the audit the Staff indicated they would respond to Consumers by March 26, 1982. The review issues involved include adoption of soil spring constants, settlement monitoring and construction dewatering for underpinning work related to the Service Water Structure.

ITEMS DISCUSSED:

1. Adoption of Soil Spring Constants. (Reference - Calculations by Bechtel DQ-29(Q) dated 12/10/81 and 2/22/82; Calculations dated 12/11/81 by G. W. Cowling; App. A entitled Service Water Pump Structure, Three-Dimensional, Finite Element Model provided by Carl Dirnbaver on 2/23/82).

   Bechtel indicated that values of soil spring constants determined in G. W. Cowling's calculation for System 5, (Long term condition) are K_{main} = 190 KCf and $Ku/_{pin}$ = 230 KCf for adopted differential settlement condition of 0.2" beneath portion on till and 0.5" beneath underpinning wall. For adopted differential settlement condition of 0.4" beneath portion on till and 0.1" beneath underpinning wall the spring constant values are k_{main} = 95 KCF and $Ku_{/pin}$ = 1150 KCf. The Staff indicated its agreement with the values of k_{main} = 190 KCf and $Ku/_{pin}$ = 230 KCf as being reasonably conservative for the design of the SWS. The Staff did not object to adoption of the other spring constant values but do not feel the differential settlement which is assumed in this condition is likely to occur.

The values of the short term soil springs for seismic loading (System 4) are not identified in Table 1, App. A but were provided in previous testimony and technical reports by Consumers. The Staff has not completed their review of the seismic spring constants and will respond to Consumers at a later date on the appropriateness of these values.

The values of spring stiffnesses adopted for system 3 (k_{main} = 150 KCF, $ku/_{pin}$ = 400 KCF) and FSAR load combination conditions are acceptable to the Staff. The Staff noted that the values adopted for system 5 (k_{main} = 190 KCF and $ku_{/pin}$ = 230 KCF) would also be a reasonable estimate of stiffnesses for FSAR load combination conditions under System 4.

There was considerable discussion on Systems 1 and 2 in Table 1, Appendi with respect to how structural stresses have been determined due to jacking loads and also due to settlements which have already occurred. The geotechnical area of concern is identifying the appropriate soil stiffnesses to use for these loading conditions. Because of the inabili to reach an agreement on these issues, it was suggested that L. McKelvey from Bechtel and S. Poulos from GEI get together by conference call next week to work out an understanding on what conditions require analysis. Following their discussions, another conference call would be arranged with the parties of today's conference call to inform both Consumers and the Staff on what agreements could be reached and what soil spring stiffnesses are required.

2. Settlement Monitoring During Service Water Structure (SWS) Underpinning. (Reference - Drawing C-2003). As agreed upon between Consumers and the Staff during the March 19, 1982 audit, a permanent bench mark will be added near the southeast corner of the SWS. Locating this new permanen benchmark on the east wall as far south as it can conveniently be posit near SW-3 is acceptable to the Staff. The major reason for this additi is to permit the differential settlement to be accurately established between the portion of the SWS founded on till and the overhanging portion presently founded on the fill.

   The Staff and its Consultants indicated their difficulties with Consume previously stated intent not to require control of underpinnning operat with established allowable settlement limits. NRC difficulties include the strain gage approach proposed by Consumers may not be a sensitive control and may not give as early a warning as measured settlements on bench marks. The Staff and its Consultants recommended that, similar t what is being carried out for the Auxiliary Building, allowable settler limits be established at the permanent benchmarks based on a structura analysis, where critical stresses due to differential settlement beyon rigid body motion have been calculated.

Consumers indicated their concerns with establishing allowable differential settlement limits which included recognition of the more rigid structure and considerably shorter length of the main portion of the SWS on till in comparison to the Auxiliary Building. These conditions, in Consumers op ion, would result in very small settlement limits which would be impractical to measure and would possibly be overshadowed by daily fluctuations due to climate changes. This matter, after considerable discussion, was not resolved but an understanding was reached that the Staff would not take a position until after their review of the strain monitoring program had been completed. This program was to be provided by Consumers in early April. Consumers agreed to consider the Staff's recommendation and provide a more definite indication on the magnitude of differential settlements which are involved.

With respect to the frequency of readings indicated in Step 3 on Drawing C-2003, the Staff made the following recommendations:

a. At least one week prior to the start of excavation below the foundation slab of the SWS, good background settlement data should be obtained for the three permanent benchmarks by increasing the frequency to a minimum of three times a day in order to observe the effects of climate changes. Consumers indicated their agreement to this Staff request.

b. When excavations below the SWS foundation slab proceed in order to install corner piers 1, 2, 3 and middle pier 4 and during the jacking of these piers, the frequency of settlement monitoring should be increased to a minimum of twice per shift. The frequency of readings can be lengthened to once every 24 hours after corner piers 1, 2 and 3 are completed and before work on pier 4 is initiated, if access to the piers is by way of an excavation outside the SWS as presently being considered by Consumers. Consumers expressed agreement with this request.

c. Following the successful construction of piers 1, 2, 3 and 4, the frequency of readings as presently proposed by Consumers on Drawing C-2003 would be acceptable to the Staff.

   It was indicated to Consumers that this increased frequency of readings during the more critical underpinning operations was also applicable to the strain gage monitoring program. Consumers agreed to consider the need for increased frequency of strain monitoring in their current work in developing this program. The frequency of readings proposed by Consumers for building settlement markers other than the permanent bench marks is acceptable to the Staff.

3. Construction Dewatering. (Reference - Plan of Construction Dewatering, Sections A-A and B-B, and Dewatering Procedure transmitted February 26, 1982 from Spencer, White and Prentis, Inc. to Bechtel Power Corporation). The Staff indicated their acceptance of the locations of the dewatering wells and piezometers (observation wells) and made the following recommendations on the above referenced information:

   a. The depths of the six proposed piezometers should extend to at least elevation 570. The top of the filter sand in the piezometers should extend to elevation 590 and then be sealed above this elevation.

   b. The type of piezometer to be installed should be sensitive (e.g., 3/8" maximum inside diameter or an air pressure cell type) to sudden piezometric changes in order to avoid long periods of time lag responses.

   c. The specification for installing and operating the construction dewatering system should establish a construction control on the upper phreatic surface. This control should require a minimum 2 foot depth between the upper phreatic surface being controlled by dewatering and the bottom of any open underpinning excavation at any given time. The depths of the proposed dewatering wells should then be drilled accordingly to accomplish this construction control on the upper phreatic surface.

   d. Placement of the filter sand in the dewatering wells should be by the tremie method rather than shoveling from the ground surface in order to avoid segregation of the filter sand particles in holes larger than 6-inch in diameter. Consumers agreed to modify the construction dewatering plans and specifications to incorporate the Staff's recommendations for above items 3a., 3b., 3c., and 3d.

   e. The Staff recommended that paragraph 7.2 of the dewatering procedures provided by Consumers be modified to define what constitutes soil particles in the discharge water (inorganic, nonmetallic materials coarser than 0.005 millimeter) and to indicate the frequency of testing similar to the agreements reached with the Staff as reflected in the June 18, 1981 letter from R. Tedesco to J. Cook. Consumers responded that the construction dewatering is a temporary system and therefore not subject to the agreements reached in the June 18, 1981 letter which covered the permanent dewatering system. There was considerable discussion on the safety significance and impact that a filter media criteria of 0.005 mm versus 0.05 mm (Consumers) could have on underpinning operations for both the Auxiliary Building and Service Water Structure. The Staff acknowledged that the 0.05 mm criteria proposed by Consumers would not result in a serious removal of foundation soil particles if this condition persisted for a period of several weeks. On the other hand the Staff considered the volume of soil fines which could be removed to be excessive if soils finer than 0.05 mm were being removed by dewatering over the anticipated one year period when underpinning work was being completed. The past monitoring procedures employed by Consumers on construction dewatering did not permit a conclusion to be reached as to whether a real problem existed for fines sized between 0.05 mm and 0.005 mm. The Staff suggested a compromise where the Staff would agree with Consumer's proceeding with construction dewatering using the 0.05 mm filter criteria but requiring the testing for soil particles at both the 0.05 mm and 0.005 mm filters. Consumers agreed to bring the results to the attention of I&E inspectors and NRR of any test where the amount of soil particles in the discharge water exceeded the limiting 10 ppm when measured on the 0.005 mm filter. At that time an engineering evaluation would be made as to the seriousness of the developing condition based on actual seepage pumping rates which are not now available. If the loss of soil particles were deemed significant enough during the remaining period of underpinning the Staff would require remedial actions to reduce that loss.

In response to Consumer's request for NRC approval in their proceeding with construction dewatering, the Staff indicated their concerns have now been resolved but that approval must come from Division of Licensing. The Staff indicated D. Hood would be notified of the results of today's conference call and suggested that Consumers directly contact him sometime later this afternoon.

On a matter not directly related to the subject of today's call, Bechtel indicated they are presently considering running a loading test at locations near, but not in the actual pier locations at both the Auxiliary Building and Service Water Structure areas. This planning is in the early stages and it is Consumers intention to submit the load test procedures and details for Staff review in the very near future.

J. Kane

3/B6

NOV 1 2 1982

Docket Nos: 50-329 OM, OL
and 50-330 OM, OL

APPLICANT: Consumers Power Company

FACILITY: Midland Plant, Units 1 and 2

SUBJECT: SUMMARY OF JULY 27 - 30, 1982, AUDIT ON SOILS REMEDIAL ACTIVITIES

On July 27-30, 1982, the NRC staff and its consultants met in Ann Arbor, Michigan with Consumers Power Company (the Applicant), Bechtel and their consultants to audit analyses, designs and preparations for remedial measures to correct the foundations and utilities on inadequately compacted fill soils at the Midland site. Meeting attendees are listed by Enclosure 1.

On July 19, 1982, the staff issued a draft of the second supplement for the Midland SER which primarily addresses the soils settlement review. A listing of the outstanding review items in this draft SSER was prepared by the applicant and served as the meeting agenda. The list was updated at the conclusion of the meeting to indicate which of those items had been included in the staff's audit. Enclosure 2 is the resulting agenda. The same-numbered items from Enclosure 2 are discussed below in this summary. Selected handouts provided during the meeting are shown as attachments within Enclosure 3.

General Items

1 - 5. Not included in Audit

6. NRC input into the final SSER will cover range of applied bearing pressures' static and dynamic loading

A draft of FSAR Table 2.5-14, including bearing pressure data for the Auxiliary Building (AB), was provided (Attachment 1). The staff reviewed the table, noted that the information was acceptable and that once provided for the docket and verified, this item would be technically closed.

7 & 8. The applicant was requested to determine that 1.5 x FSAR seismic response spectra analyses are conservative for the auxiliary building (AB), service water pump structure (SWPS), and borated water storage tank (BWST) in comparison to site-specific response spectra (SSRS).

The applicant has not provided comparative plots of floor response spectra that were requested by the staff for all buildings (seismic margin review).

The NRC structural engineering staff reviewed calculations at 5 points of elevation for the AB to determine if 1.5 x FSAR response

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spectra enveloped the results obtained by using the SSRS. For these five points, the floor response spectra generated by the use of 1.5 times the FSAR spectra enveloped the respective floor response spectra developed from SSRS. Additional locations in this and other structures will be addressed as part of the seismic margin study.

The applicant also noted that the use of the floor response spectra derived from the seismic margin earthquake would be according to the seismic margin review criteria submitted to the staff by letter of September 25, 1981. The results of the seismic margin review will be submitted to the staff during the first quarter of 1983.

9. Test data on #9 and #10 Fox-Howlett rebar splices with up to 2% strain

Copies of test data up to 2% strain for #9 and #10 Fox-Howlett rebar splices were provided to the NRC during the audit. Copies were also sent to the NRC consultant, Science Applications Institute by letter dated July 16, 1982.

The NRC found the information acceptable after preliminary review. Pending subsequent NRC discussions with its consultant, this item may be closed.

10. Identification, inspection, and repair procedures for concrete crack repair

Criteria for concrete cracks were agreed upon and will be documented by the applicant in a letter in early August 1982 (Post script: see applicant's letter of August 2, 1982).

The crack repair program applies to the DGB, SWPS, Control Tower and Electrical Penetrations Areas of the Auxiliary Building and Feedwater Isolation Pits, which will be completed prior to the first refueling of the plant. It consists of the following three points:

(1) Repair by epoxy injection any cracks in the structures which are below the permanent ground water table and which exhibit weeping characteristic. This repair will be performed from the inside of the structures.

(2) Coat the splash zone of the exterior surface of the south wall of the Service Water Pump Structure which is in contact with cooling pond water with waterproofing compounds. The waterproofing compound will be one of the three compounds recommended by consultants in their report "Effects of Cracks on Serviceability of Structures in the Plant".

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(3) Repair by epoxy injection existing cracks which are 20 mils and larger and apply a sealant to the surfaces of the concrete walls in the following accessible areas (i.e. areas where removal of soil or installed equipment or installed components is not necessary to perform the repair). The extent (length) of the crack that will be injected with epoxy will include at least that portion with crack width of 10 mils or larger.

Prior to the initiation of repairs, all cracks 20 mils and larger and weeping cracks in the applicable areas will be identified. A verification of 'his identification to a tolerance of +5 mils will be performed. This verification and subsequent will be in accordance with the quality program. The material for structural epoxy adhesive will be "concresive-1380" manufactured by Adhesive Engineering Company, or equivalent.

The areas to be repaired for each applicable building are as follows:

DGB

(a) All accessible interior reinforced concrete walls.
(b) All accessible exterior concrete walls.

CT&EPAs

(a) All accessible exterior concrete walls.

SWPS

(a) All accessible exterior walls.

11 & 12. Not included in audit.

Auxiliary Building

1. Resolution of allowable vertical differential settlement and strain that will stop underpinning construction and require installation of temeporary supports

The NRC staff reviewed the allowable settlement calculation resulting from analysis of the construction condition using a subgrade modules of 70 KCF and analysis of reduced support along the EPA due to tunneling (Attachment 4).

Attachment 2 provides definitions of "alert", "action" and "requalify" levels which were agreed upon for underpinning activities. Attachment 3 provides numerical values which were agreed upon. The levels apply to Phases II, III, and IV.

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This item was accepted by the staff.

2. Compaction control specification for granular fill beneath feedwater isolation valve pits (FIVPs)

It was agreed that the fill beneath the FIVP will be tested using the procedures outlined in the Seabrook FSAR. A copy of a similar FSAR section was provided by the NRC. It was also agreed that the fines portion of the fill shall be non-plastic. This will be verified by the resident geotechnical engineer by appropriate testing (hydrometer of Atterberg limits). The backfill will be properly moisture conditioned by soaking immediately prior to compaction. The soaking means will be approved by the resident geotechnical engineer. Compaction acceptance criteria will be 95% modified proctor or 85% relative density (whichever testing standard results in the maximum dry density) based on tests performed prior to placement. The applicant also committed to performing a laboratory compaction or relative density test to establish maximum dry density on soil material taken from each field density test location. Bechtel compaction control specification will be revised.

Additional compaction equipment (e.g. self propelled double drum compactor) will be qualified by the test fill method.

3. Methodology for transferring final loads to permanent underpinning wall

Preliminary copies of Mergentime/Hanson Drawings S-74 and S-74a (see SSER #2, Appendix I) not yet reviewed by Bechtel, were provided for staff review. Analysis of the permanent wall and preliminary design details were also reviewed. The review included methodology, rebar stresses in critical areas, and connection to existing structure. The staff found these items to be acceptable.

The transfer of loads will be accomplished by the use of hydraulically actuated steel jacks that are incrementally increased to the specific loads determined by the structural analyses. When the predetermined loads have been developed by the jacks, the loads will be maintained and locked off provided that the following criteria are met:

(1) The pier will be loaded to 125% of its specified jacking load and continued at the load until the relative movement between the top of the pier and the underpinning structure is less than 0.01 in. in a continuous 1 hour period. When this condition is satisfied,

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(2) The pier load will be reduced to 110% of its specified jacking load and continued at that load until the relative movement between the top of the pier and the underpinning structure is less than 0.01 in. for a continuous 24-hour period. When this condition is satisfied, the pier will be locked off.

(3) Jacking loads for the permanent underpinning will be maintained at the specified value for at least 30 days.

(4) A semilogarithmic plot of settlement versus time will be developed to allow determination of when secondary consolidation has been reached.

(5) The settlement increment in the last 30 days of sustained load will not exceed 0.05 in.

(6) The settlement in the last 10 days of sustained load will not exceed 0.01 in.

(7) Wedges to be used for the permanent wall will be driven tight and permanently welded in place. In case a predicted jacking load is not obtained (when a 0.03-in. upward movement of the existing structure occurs) jacking loads should be reduced to 80% of the load at which the movement occurred and this load will be used in the analyses to determine subsequent jacking loads.

4. Updated scope of construction for Phases III and IV

The plan which describes the construction scope (Drawing 7220-SK-C-0101) (see SSER #2, Appendix I) was reviewed. A discussion was also held regarding construction sequence. The staff found these matters to be acceptable.

5. Resolution of pier and plate load test details on maximum test load, locations, and time for performing test

The load test will be performed on Pier W-11. The proposed load sequence is to jack the load from 0 to 50% of the bearing pressure allowed for the seismic loading combination, then decrease the load to 25%, and then increase the load to 130%. The staff agreed that no additional plate load test is required. The staff found these details to be acceptable.

6. Long-term settlement monitoring plan during plant operation

   This is a technical specifications item. The information will be provided to the NRC as part of the FSAR technical specification submittal in October 1982.

7. FSAR documentation on as-built conditions

   This is a confirmatory item which will provide the level of construction information typical of an FSAR. The information will be provided to the NRC once the appropriate construction stage has been achieved.

8. Design modification at freezewall crossing with duct banks

   The applicant had previously committed to provide a report addressing the installed surcharge loading program, monitoring results and backfill techniques. The proposed method for backfilling monitoring pits will be provided prior to accomplishing the work. This carryover item from earlier meetings continues as a confirmatory issue.

9. Resolution of required depths of construction dewatering wells

   The applicant agreeds with a staff position that, when excavating in cohesionless (natural or fill) soils, the groundwater will be maintained 2 feet below the advance of excavation.

   In addition, a probing program will be used in selected piers. As a minimum, these piers include E12, W12, E10, W10, E7, W7, E4, W4, CT1, CT6, and CT12. Test holes between 1 in. and 4 in. in diameter will be advanced to a depth of 5 ft beneath the proposed bearing level (from a level 5 ft above the bearing level) in these 11 selected piers to determine whether groundwater under pressure exists in sufficient volume to require special pier dewatering. It water pressures are low, excavation to the bearing level will continue. If water pressures are shown to be high in the test holes, special dewatering (e.g., wellpoint or other suitable means) will be used to lower the water table at that pier to at least 2 ft below the bearing level. The hole beneath the final bearing level will be grouted. Although the available information indicates that the bearing stratum is a fairly homogeneous hard clay, it is possible that special pier dewatering will be needed. These holes will be used

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by the applicant as a conservative measure to confirm subsurface conditions before the bearing level is reached. Interpretation will be done by the resident geotechnical engineer. This item is acceptable to the staff on this basis.

10. Monitoring matrix showing allowable settlements and strains

An updated copy of the monitoring matrix (Bechtel Drawing 7220 C-1493(Q), Rev. 1) (Attachment 7) was provided. Alert, action and requalify levels will be added as agreed above (AB Item 1).

The staff agreed that no alert or action level needs to be established for monitoring strain. However, the strain data are considered supplementary to understand the behavior of the building and strain levels greater than 0.0010 in/in. are a factor to be considered in the raising of the alert and action settlement levels. This item is acceptable to the staff on this basis.

11. Electrical penetration area (EPA) and control tower (CT) relative horizontal movement criteria

The NRC staff reviewed drawings showing the gap detail between the EPA/CT and the turbine building (TB). The minimum gap between structural members of the CT and TB is 8 in.; the minimum gap between structural members of the EPA and TB is 6 in.

The staff agreed that no acceptance criteria will be required for horizontal movement during underpinning. Data from the horizontal instrumentation measurements will be recorded and used as supplementary information to the differential settlement records in the overall evaluation of structure movement during underpinning work.

12. Changes in pier configuration

The applicant has determined that piers CT4X and CT9X located along Column line K_c at 5.9 and 7.2 will not be required. Piers will be required at H_k and 5, and at H_k and 8. The NRC staff reviewed Bechtel Drawing 7720-SK C-0101 (Rev. 0) and Mergentime/Hanson drawing S-74 (Rev. 2) showing the details of these piers (see SSER #2, Appendix I). This is acceptable to the staff.

13. Details on stiffened bulkhead during drift excavation

The NRC staff reviewed and agreed with the calculations of the drift/stiffened bulkhead design. The staff also agreed to constructing the drift support system in 2-foot increments, with lagging and tight backpacking completed up to the bottom of the EPA foundation slab and with an excavation bench on the FIVP side.

14. As-built plan for deep-seated benchmarks

The NRC staff reviewed Bechetel Drawings 7220-C-1490 and C-1491 (Attachment 7) showing as-built locations of the AB deep-seated benchmarks and found them to be technically acceptable.

15. Review of Specification 7220-C-200, Emergency Actions

The flow charts for the emergency actions of Specification 7220-C-200 were reviewed in detail. The staff found the flow charts to be acceptable.

Service Water Pump Structure

1. Complete staff review of sliding and lateral soil pressure calculation under dynamic loading

The NRC staff completed review of the sliding and lateral soil pressure calculation. Seismic loads equal to 1.5 times the FSAR SSE loads were used and were found to exceed SSRS loads. Factors of safety against sliding were 1.45 (N-S direction) and 1.50 (E-W direction), which exceed the staff's minimum requirement of 1.1. This technical item is closed.

2. Resolution of pier and plate load test details on maximum test load, locations, and time for performing test

The load test will be performed on Pier 1 (east side). The proposed load sequence is to jack the load from 0 to 50% of the bearing pressure allowed for the seismic loading combination, then decrease the load to 25%, and then increase the load to 130%. The staff agreed that no plate load test will be required. This technical item is closed.

3. Resolution of required depths of construction dewatering wells

For monitoring of construction dewatering at the SWPS, 12 piezometers will be provided. Six will be sealed in the zone from el 570' to el 590'. Soil sampling will be continuous from el 570' to el 585' in borings at the location of the six perimeter piezometers. The other six will be installed at the subcontractor's discretion.

The water surface will be maintained 2 feet below the bottom of pier excavations if sand is present within 8 ft of the pier foundations as indicated by the continuous sampling in the six perimeter piezometers. If sand layers are identified in the exploratory borings for the piezometer installations, the wells will be lowered to maintain the 2 foot requirement. The results of the explorations and the final installation depths of the dewatering wells are to be provided to the NRC staff when available. This technical item is closed.

4. Methodology for transferring loads from jacks to permanent wall and locking off

Drawing 7220-C-2035-Q Rev. 2, with the relevant parts of Specification 7220-C-194 showing final load transfer procedures, were reviewed by the NRC staff and found to be acceptable. This technical item is closed.

5. Long-term settlement monitoring plan during plant operation

This is a technical specification issue. The information will be provided to the NRC as part of the FSAR technical specification submittal in October 1982.

6. FSAR documentation on as-built conditions

This is a confirmatory item with technical issues resolved. The information will be provided to the NRC once the appropriate construction stage has been achieved.

6a. Strain monitoring to measure acceptable allowable strain

The NRC staff's evaluation of the applicant's June 14, 1982, submittal indicated the proposed 5/16 inch displacement (extension) criterion over a single 20-foot gage length was not acceptable and the staff recommended that several gages of shorter lengths be installed to permit identification of the more highly stressed sections. In the meeting of June 25, 1982, the applicant committed to using four 5-foot long gages in place of or in addition to the single 20-foot gage. The action and alert limits for the 5-foot long gages will be based on the yield strain of the reinforcing steel.

7. Staff input into the final SSER will describe computed earth pressures under both static and dynamic loading and design methods

Review of computed earth pressures was completed. This technical item is closed.

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8. The NRC staff is to review and evaluate the applicant's analysis as identified in response to Request 2.8 of Enclosure 8, NRC letter dated 5/25/82 (interaction of circulating water and SWPS wall).

The NRC staff reviewed the drawing showing the structural gap between the circulating water intake structure (CWIS) and the SWPS, and compared this gap with the predicted deflections for each structure under earthquake loads. The 1 in. minimum gap is sufficient to accomdate the relative calculated gap of 0.518 in. Simarily, the 1 in. gap between the SWPS and the cooling pond retaining wall accomodates the calculated relative gap of 0.25 in. during a SSE. This item is closed.

9. Check dowels for shear and tension capability

The staff reviewed the design calculations, discussed the design methodology, and determined the shear and tension capability of connections for the underpinning to the existing structure. The items were found to be acceptable. This item is closed.

Borated Water Storage Tank

1. Long-term settlement monitoring plan during plant operation

This is a technical specification issue. The information will be provided to the NRC as part of the FSAR technical specification submittal in October 1982.

2. FSAR documentation on as-built conditions

This is a confirmatory item with technical issues resolved. This information will be provided to the NRC once the appropriate construction stage is achieved.

3. Staff calculational review for governing loading combinations in structural design

The NRC staff reviewed the calculation for design of the new ring beam foundation for applicable load combinations. The governing load combination is:

$U = 1.4D + 1.4T + 1.4F + 1.7L + 1.7H + 1.9E$
where component loads are identified by FSAR Section 3.8.6.3.1.

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The staff also reviewed the methodology used for design of a typical section considering forces and moments and found it to be acceptable. Additional information of a confirmatory nature will be provided as part of the seismic margin study to demonstrate the adequacy of use of 1.5 times the FSAR response spectra relative to use of SSRS.

Underground Piping

1. Staff evaluation of previously submitted reports on underground piping not completed

The NRC staff and its consultant from ETEC reviewed the calculations for stresses due to seismic and settlement effects. The staff agreed with the assumptions, methodology, and results of the analyses.

The staff completed its geotechnical review of previously submitted reports. The applicant agreed to add five additional settlement and strain monitoring stations as requested, plus settlement markers at each end of transition zones of replaced/rebedded pipes as shown on Drawing 7220-SK-C-745 (see SSER #2, Figure 2.11). The five addi- tional settlement and strain marker locations are station 1 + 32 and 3 + 15 for line 26"-OHBC-15; station 1 + 55 for line 26"-OHBC-20; station 0 + 80 for line 26"-OHBC-55 and station 3 + 00 for line 26"-OHBC-54. The The applicant also agreed to change the monitoring frequency to once per month for the first 6 months of plant operation. The frequency of readings will be lengthened to the 90 day interval following the intial six month period if the settlement readings have stabilized (not larger than 0.10 inch change from the previous reading). This will be written into the technical specifications. This item is closed.

2. The applicant's proposed reinstallation of 26-inch and 36-inch diameter pipes including review of analysis, properties of backfill, extent of excavation, details of transition, and controls during consturction

The staff consultant visited the site and observed the arrangement of the service water piping in the SWPS.

The design approach for reinstallation of the service water pipe was reviewed and approved. The applicant provided a preliminary stress summary table for the piping to be reinstalled. The final table will be provided by August 20, 1982. Drawing 7220-SK-C-745 was marked to show the settlement and strain monitoring locations that were agreed upon.

Properties of the proposed backfill were provided for review. It is planned to use a mixture of sand, cement, and fly ash. The commercial name of this product is "K-Krete" (Attachment 6).

The next FSAR revision will document the design for the reinstalled piping, properties of the backfill material, and the stress summary table. This item is closed.

3 & 5. Plant control restricting placement of heavy loads over buried piping and conduits

Technical specification proposal by applicant for long-term settlement and strain monitoring plan during plant operation

These are technical specification items. The information will be provided to the NRC as part of the FSAR technical specification submittal.

4. FSAR documentation on as-built conditions

This is a confirmatory item with all technical issues resolved. The information will be provided to the NRC once the appropriate construction stage is acheived.

Diesel Generator Building Analysis

1. Resolution of assumptions (structural rigidity) and completion of analysis that uses correct settlement values; documentation of these results with comparison to recorded and predicted settlements

The NRC staff reviewed calculations for the diesel generator building which included settlement effects prior to, during, and after surcharge, including predicted values for the life of the plant.

The maximum calculated stress for the period March 28, 1978, to August 18, 1978, is approximately 11 ksi.

The NRC staff expressed the need to further review the results of calculations on the effects of settlement on the DGB including the method used by the applicant to characterize the shape of the structure resulting from actually recorded settlements and predicted settlement values.

Bearing pressures were reviewed and found to be acceptable.

2. Long-term settlement monitoring plan during plant operation

This is a technical specification item. The information will be provided to the NRC as part of the FSAR technical specification submittal.

Permanent Dewatering

1. Resolve availability of 60-day period in view of recharge rate in wells in AB railroad bay area

   The applicant reviewed with the NRC staff the events related to the rupture of a construction water pipe which affected the recharge response in the railroad bay area.

   Information in response to written questions by NRC Hydraulic Engineering Section were provided for future review in Bethesda and included information on the period to initiate shutdown. This period will be documented in the technical specifications. A report will be submitted after system installation to document the water contours developed by the permanent dewatering system. This report will provide verification of any water source in the railroad bay area.

2. Requirements of permanent dewatering system during plant operation

   This is a technical specification item. The information will be provided to the NRC as part of the FSAR technical specification submittal.

3. Results of typical well fines monitoring

   The applicant provided typical results from the July fines monitoring of the AB construction dewatering wells.

Well	5 micron (ppm)	50 micron (ppm)
ME-7	0.5	0.2
ME-8	1.1	0.4
ME-9	0.5	0.3
ME-46	0.6	1.0

   This item is closed.

Other Items

A presentation was given on the project organization and consultants for the soils work (Attachment 5).

* Kane & Rinaldi comments incorporated DSH

/s/

Darl S. Hood, Project Manager
Licensing Branch No. 4
Division of Licensing

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Enclosure 2 7/30/82

O - Open Item
CON - Confirmatory Item
TS - Operating License Technical Specification
R - Technical Resolution Staff Input Pending
C - Closed Item

MIDLAND PLANTS UNIT 1 AND 2
REVIEW OF DRAFT SER, SUPPLEMENT NO 2

	SSER STATUS	AUDIT ITEM
GENERAL ITEMS		
1. Staff's input for the final SSER will include summary of subsurface investigations.	R	No
2. Staff's input into final SSER will describe laboratory and field testing.	R	No
3. Staff's input into the final SSER will include staff evaluation of pertinent soil profiles sectional views.	R	No
4. Summerize the settlement history of Catagory 1 structures other than the AB & SWPS.	R	No
5. Long term settlement monitoring plans during plant operation for other structures.	TS	No
6. NRC's input into the final SSER will cover range of applied bearing pressures static and dynamic loading.	R	Yes
7. Applicant was requested to determine that 1.5 x FSAR seismic response spectra analyses are conservative for the auxiliary building, SWPS, and BWST in comparison to site specific response spectra.	CON	Yes
8. Applicant has not provided comparative plots of floor response requested by the staff for all buildings (seismic margin review).	O	Yes

		SSER STATUS	AUDIT ITEM
9.	Test data on #9 and #10 Fox Howlett with up to 2% strain.	CON	Yes
10.	Identification, inspection and repair procedures for concrete crack repair.	CON	Yes
11.	Use of concrete expansion anchors to attach piping and equipment to masonry walls is disallowed by Staff criteria (non-soils).	O	No
12.	Staff's input into the final SSER will summarize geotechnical engineering review efforts and SHAKE computer code studies.	R	No

AUXILIARY BUILDING

		SSER STATUS	AUDIT ITEM
1.	Resolution of allowable vertical differential settlement and strain that will stop underpinning construction and require installation of temporary supports.	O	Yes
2.	Compaction control specification for granular fill beneath FIVP's.	O	Yes
3.	Methodology for transferring final loads to permanent underpinning wall.	O	Yes
4.	Updated scope of construction for Phases 3 and 4.	O	Yes
5.	Resolution of pier [illegible] test details on maximum test load, locations and time for performing test.	O	Yes
6.	Long term settlement and strain monitoring plan during plant operation.	TS	Yes
7.	FSAR documentation on as-built conditions.	CON	No
8.	Design modification at freezewall crossing with duct banks.	CON	No
9.	Resolution of required depths of construction dewatering wells.	CON	Yes
10.	Monitoring matrix showing allowable settlements and strains	CON	Yes
11.	EPA and CT relative horizontal movement criteria	CON	Yes
12.	Changes in pier configuration	CON	Yes
13.	Details on stiffened bulkhead during drift excavation	CON	Yes
14.	As built plan for deep seated benchmarks	CON	Yes
15.	Review of emergency actions C-200	CON	Yes

		SSER STATUS	AUDIT ITEM
SERVICE WATER PUMP STRUCTURE			
1.	Complete Staff review of sliding and lateral soil pressure calculations under dynamic loading.	CON	Yes
2.	Resolution of pier and plate load test details on maximum test load, locations, and time for performing test.	CON	Yes
3.	Resolution of required depths of construction dewatering wells.	O	Yes
4.	Methodology for transferring loads from jacks to permanent wall and locking-off.	O	Yes
5.	Long term settlement and strain monitoring plan during plant operation and program for monitoring horizontal movement.	TS	Yes
6.	FSAR documentation on as-built conditions.	CON	No
6a.	Strain monitoring to measure acceptable allowable strain.	CON	Yes
7.	Staff's input into final SSER will describe computed earth pressures under both static and dynamic loading and design methods.	R	Yes
8.	Staff to review and evaluate Applicant's analysis as identified in response to Request 2.8 of Enclosure 8, NRC letter dated 5/25/82. (interaction of circ water & SWPS walk)	CON	Yes
9.	Check dowels for shear and tension capability.	CON	Yes

		SSER STATUS	AUDIT ITEM
BORATED WATER STORAGE TANK			
1.	Long term settlement monitoring plan during plant operation.	TS	No
2.	FSAR documentation on as-built conditions.	CON	No
3.	Staff calculational review for governing loading combinations in structural design.	CON	Yes

mi0782-0048a168

UNDERGROUND PIPING

		SSER STATUS	AUDIT ITEM
1.	Staff's evaluation of previously submitted reports on underground piping not completed.	R	Yes
2.	Applicant's proposed reinstallation of 26-inch 36-inch diameter pipes including review of analysis, properties of backfill, extent of excavation details of transition, controls during construction.	O	Yes
3.	Plant control restricting placement of heavy loads over buried piping and conduits.	TS	No
4.	FSAR documentation on as-built conditions.	CON	No
5.	Tech Spec proposal by Applicant for long term settlement and strain monitoring plan during plant operation.	TS	No

	SSER STATUS	AUDIT ITEM
DIESEL GENERATOR BUILDING ANALYSIS		
1. Resolution of assumptions (structural rigidity)and completion of analysis that uses correct settlement values. Documentation of these results with comparison to recorded and predicted settlements.	O	Yes
2. Long term settlement monitoring plan during plant operation.	TS	No

		SSER STATUS	AUDIT ITEM
PERMANENT DEWATERING			
1.	Resolve availability of 60 day period in view of recharge rate in wells in railroad bay area of Auxiliary Building.	O	Yes
2.	Requirements on permanent dewatering system during plant operation.	TS	No
3.	Results of typical well fines monitoring	CON	Yes

mi0782-0048a168

Enclosure 3

Selected Handouts for

July 27-30, 1982, Audit

OFFICE ▶							
SURNAME ▶							
DATE ▶							

Rec'd 7/30/82

attachment 1
sheet 1 of 4

FOR INFORMATION ONLY

7220

RS-003-03

TABLE 2.5-14

SUMMARY OF CONTACT STRESSES AND ULTIMATE
BEARING CAPACITY FOR FOUNDATIONS
SUPPORTING SEISMIC CATEGORY I AND OTHER SELECTED STRUCTURES | 18 | 44

consider - change to comparison w/ gross

Unit	Supporting Soils	Foundation Elevation	Contact Stress Beneath Footing (lb/ft²): Gross Dead and Live Load	Net Dead and Live Load	Gross Dead, Live, and Seismic Load	Net Dead, Live, and Seismic Load	Net Ultimate Bearing Capacity (lb/ft²)	Factor of Safety: NET Dead and Live Load	NET Dead, Live, and Seismic Load
Category I Structures									
Reactor containment buildings	Very stiff to hard natural cohesive soils	582.5	10,000	3,300	19,500	12,800	45,000	13.6	3.5
Auxiliary building area(1) A	Very stiff to hard natural cohesive soils	562	7,000	—	8,200	1,000	45,000	NA	45.0
Auxiliary building areas B and C(1)	Very stiff to hard natural cohesive soils	579	6,600	400	10,200	4,000	45,000	112	11.3
Auxiliary building Area D(1)	Very stiff to hard natural cohesive soils	556	15,000	13,400	20,600	19,000	45,000	3.4	2.4
Auxiliary building Areas E and F(1)	Very stiff to hard natural cohesive soils	571	11,000	4,300	19,800	13,100	45,000	10.5	3.4
Auxiliary building Area G(1)	Zone 2(3)	630.5	1,400	1,000	3,400	3,000	15,000	15.0	5.0
Auxiliary building Area H(1)	Zone 2(3)	610	1,400	NA	5,100	2,200	30,000	NA	13.6
Auxiliary building Areas I and J(1)	Very stiff to hard natural cohesive soils	569	6,800	0	9,200	2,400	45,000	NA	18.8

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Attachment 1
sheet 2 of 4

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RS-003-03

TABLE 2.5-14 (continued)

Unit	Supporting Soils	Foundation Elevation	Contact Stress Beneath Footing (lb/ft²)				Net Ultimate Bearing Capacity (lb/ft²)	Factor of Safety	
			Gross Dead and Live Load	Net Dead and Live Load	Gross Dead, Live, and Seismic Load	Net Dead, Live, and Seismic Load		Net Dead and Live Load	Net Dead, Live, and Seismic Load
Auxiliary building Areas K and L(1)	Very stiff to hard natural cohesive soils	579	(2)	(2)	(2)	(2)	(2)	(2)	(2)
Feedwater isolation valve pit	Structural sand backfill	601	4,200	(4)	10,100	5,800	25,000	(4)	4.3
Diesel generator building	Zone 2(3)	628	4,400	3,600	5,700	4,900	14,000	3.9	2.9
Diesel generator pedestal foundation	Zone 2(3)	628	1,670	900	2,050	1,300	8,000	8.9	6.2
Borated water storage tank	Zone 2(3)	629	2,000	1,400	4,600	4,000	12,000	8.6	3.0
Service Water Pump Structure									

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RS-003-03

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TABLE 2.5-14 (continued)

			Contact Stress Beneath Footing (lb/ft²)					Factor of Safety	
Unit	Supporting Soils	Foundation Elevation	Gross Dead and Live Load	Net Dead and Live Load	Gross Dead, Live, and Seismic Load	Net Dead, Live, and Seismic Load	Net Ultimate Bearing Capacity (lb/ft²)	Dead and Live Load	Dead, Live, and Seismic Load
Circulating water isolation system	Very stiff to hard natural cohesive soils and dense natural sands	596.5	4,030	3,800	4,090	3,900	25,000	6.6	6.4

44

Note: Factor of safety is defined as the ratio of net ultimate bearing capacity to net contact stress beneath footing.

(1) Refer to Figure 2.5-47 for auxiliary building areas.

(2) ~~Revised values are to be provided by amendment following reanalysis.~~ 18

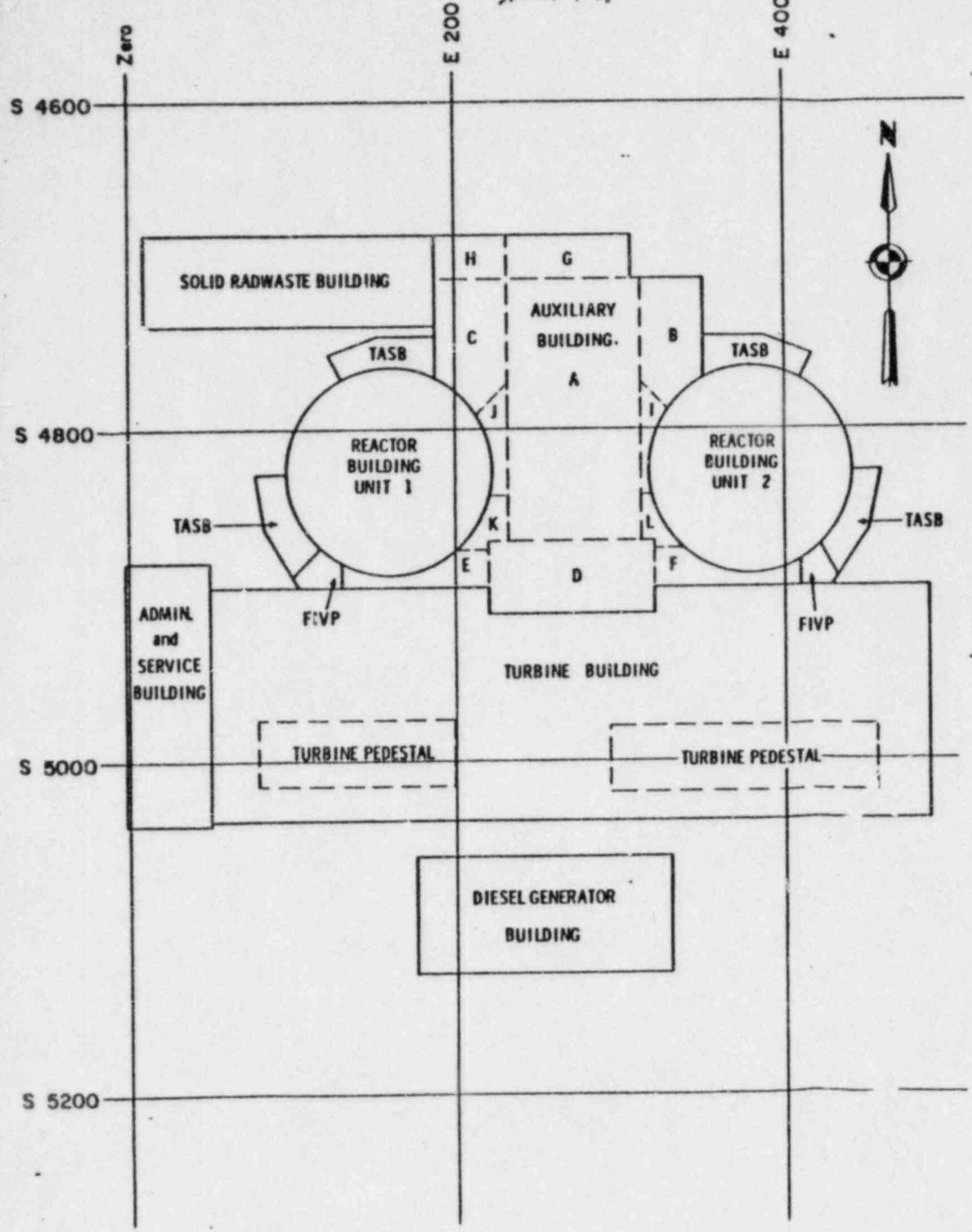
(3) Refer to Table 2.5-10 for description of Zone 2 soil.

(4) For these cases, the applied loads are less than or about equal to the depth of embedment times the unit weight of the soil. Therefore, net loads are negative or insignificant and the factor of safety against bearing capacity failure is not applicable. 44

2. LOAD IS TRANSFERRED TO AREAS D, E & F AS A RESULT OF THE UNDERPINNING OPERATION. (FROM K & L)

5. GROSS SOIL PRESSURE UNDER THE AREAS A THRU L ASSUME THE WATER TABLE IS AT EL. 585'-0.

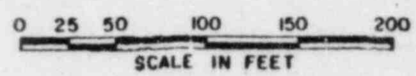
Attachment 1
Sheet 4 of 4



		El_m	P_s	P_{n1}	P_{n2}
AREA FILL LOAD		603	4.03	4.03	2.53
SOLID RADWASTE BLDG		629.5	4.9	4.9	4.9
AUXILIARY BLDG	A	562	9.7	6.66	2.60
	B & C	579	2.9	1.04	-1.95
	D	609	5.3	5.3	4.18
	E & F	609	6.0	6.0	4.88
	G	630.5	0.9	0.9	0.9
	H	610	0.9	0.9	-0.16
	I & J	569	1.4	-1.15	-4.77
	K & L	579	0.8	-1.06	-4.05
REACTOR BLDGS 1 & 2		582.5	10.0	8.39	5.61
TURBINE BLDG.		609	3.0	3.0	1.88
TURBINE PEDESTALS (2)		602	5.0	4.87	3.31
DIESEL GENERATOR BUILDING		628	SEE	NOTE	6
FEEDWATER ISOLATION VALVE PIT (FIVP)		616.0	1.5	1.5	0.81
TENDON ACCESS SHAFT BUTTRESS (TASB)		587.5	1.0	-0.27	-2.73
ADMINISTRATION AND SERVICE BUILDING		629.5	4.5	4.5	4.5

NOTES:

1. El_m is the elevation of the bottom of the foundation,
2. P_s is the gross structural load.
3. P_{n1} is the net load intensity before the cooling water reservior filling $P_{n1} = P_s$ - Excavation load (corrected for bouyancy).
4. P_{n2} is the net load intensity after the cooling water reservior filling $P_{n2} = P_{n1}$ - Hydrostatic pressure.
5. All units for load intensity in kips per foot square (ksf), elevations in feet from U. S. G. S. datum.
6. $P_s = P_{n1} = P_n = 3.0$ ksf was used for the diesel generator building load and 2.0 ksf was used for the surcharge load for determining the influence on the power block structures only.



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RS-003-C3

ATTACHMENT TO
CALC. NO. DQ 67 (9) Rev 0

SH 4

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Soil Pressures Used
In Settlement Analysis
of Power Block
(SK-G-59, Rev 2)

FSAR Figure 2.5-47

Attachment 2
Sheet 1

Alert Level

All values up to the alert level are considered to be within normal working ranges.

Settlement readings should be reviewed by the resident structural engineer daily. In general, for readings below the alert level, attention should be focused on the value of the readings versus the construction progress and any indication of trends that would indicate the alert level will be exceeded.

Once the alert level is exceeded, the site resident engineer must inform engineering in Ann Arbor of the situation. The data including information from the other appropriate data mechanisms should be evaluated in total. Where trends exist that indicate the action level is likely to be reached, plans should be evaluated to remedy the situation. (Note: It is recognized that the evaluation may well conclude that no changes are warranted.)

Action Levels*

Differential settlement ~~Values~~ values which reach ~~in excess of~~ the action level must be reviewed by the resident structural engineer and as soon as possible by engineering in Ann Arbor.

Plans, and actions described in Specification C-200 should be initiated to modify the condition that caused the settlement reading to ~~exceed~~ reach the action level. Consumers Power Company must be informed of the revised plan so that the NRC can be advised of the situation. The revised plan shall be initiated immediately upon verbal notification by the resident structural engineer. ~~(Note: It is recognized that the evaluation may well conclude that no changes are warranted.) If continuous movement beyond action level occurs, immediate action shall be taken per Specification C-200.~~

Requalifying Level

~~(Requalify Level),~~

If the differential settlements reach 0.50 inch, the applicant will start discussions with NRC for consideration of and concurrence with future actions before implementing those actions.

* - Cracking levels correspond to these definitions for Alert and Action.

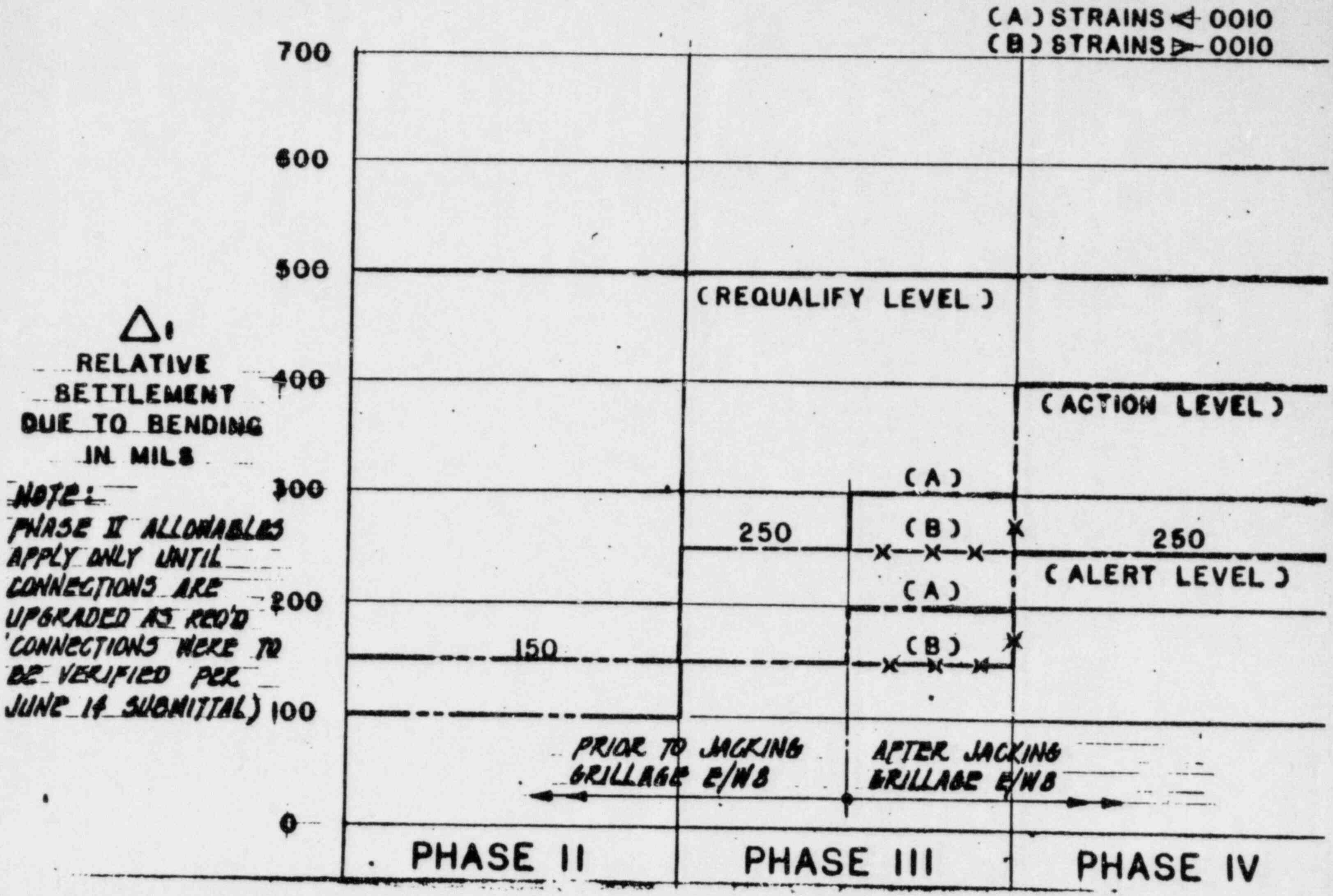
/dj

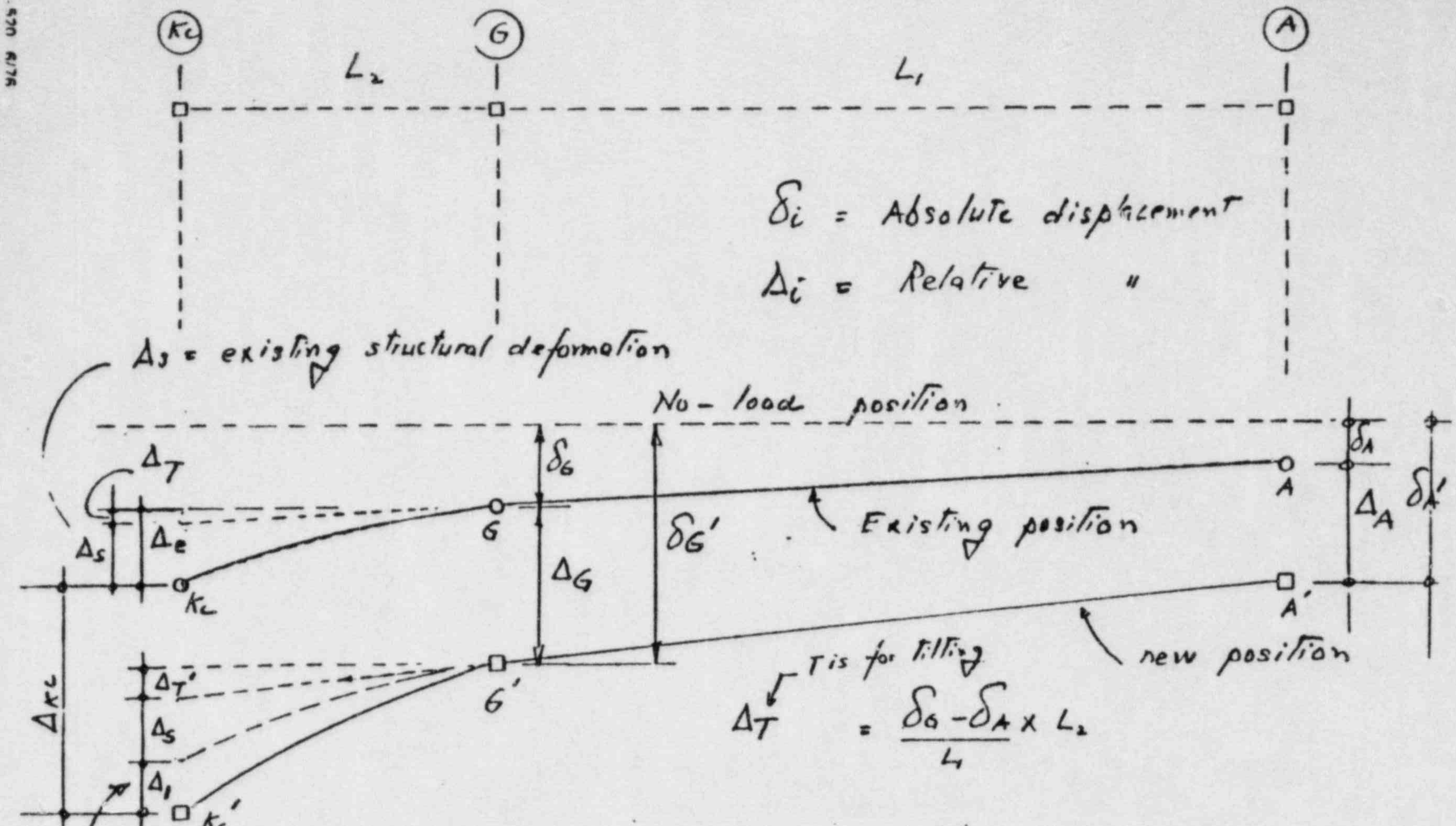
072801

Attachment 3
Sheet 1

REMEDIAL SOILS

SETTLEMENT MONITORING MATRIX





$$\Delta_{allowable} = \left\{ \Delta_{KC} + \cancel{\Delta_s} + \Delta_T \right\} - \left\{ \Delta_G + \Delta_T' + \cancel{\Delta_s} \right\}$$

$$= \Delta_{KC} + \Delta_T - \Delta_G - \Delta_T'$$

7/29/82

Attachment 4

CALCULATED DISPLACEMENTS
AT DEEP SEATED BENCHMARKS

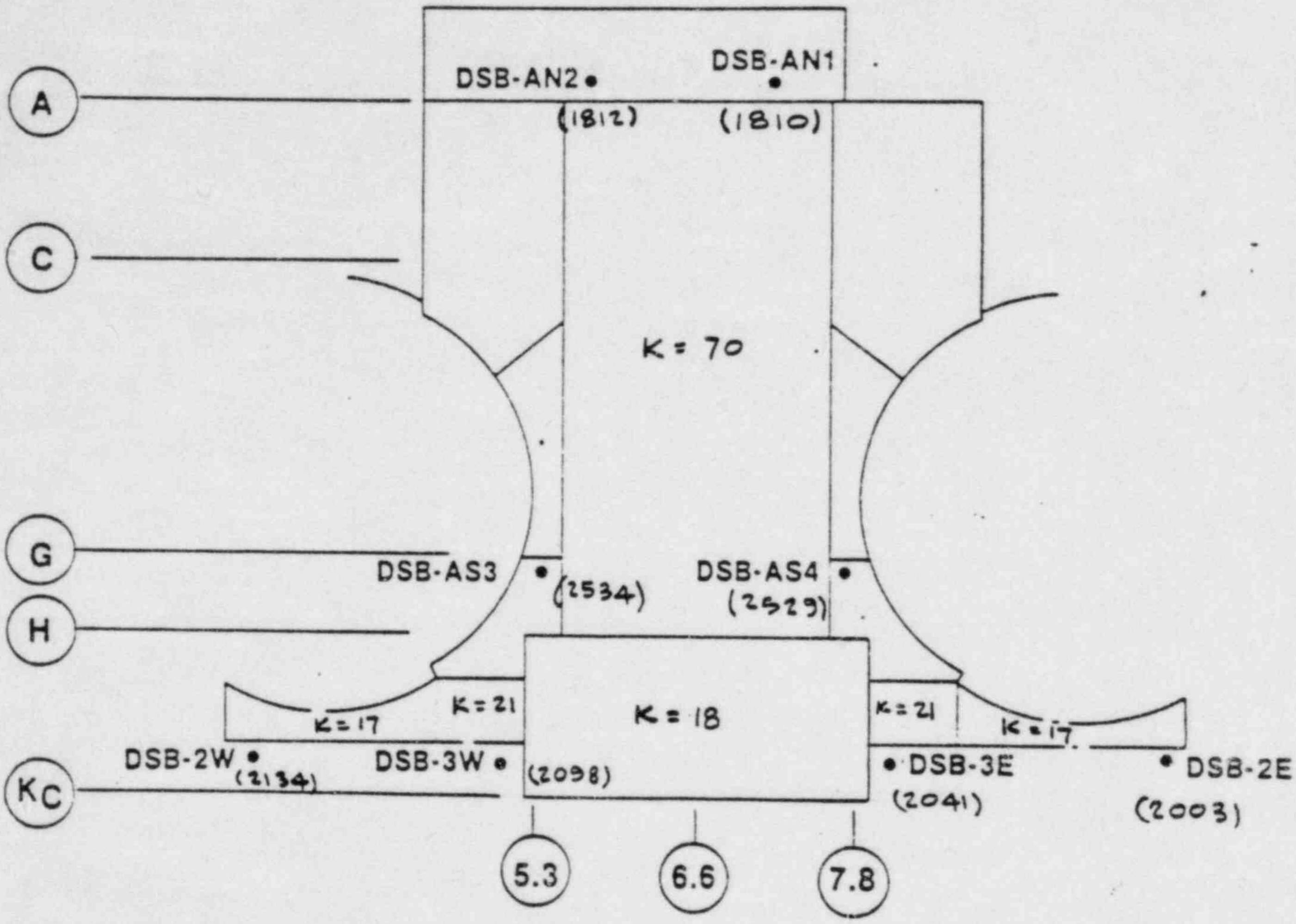


Figure 1

*Exact locations are shown on drawings C-1490 and C-1491

G-2516-02

Attachment 4

DISPLACEMENTS WITH K = 70 KCF

NODE	BENCH MARK	EXIST^NG 1	STAGE 1 2	3	STAGE 2 4	5	STAGE 3 6	7	
1810	DSB-AN1	-1.03"	-0.974	-1.037	-1.007	-1.120	-0.560	-0.854	
1812	DSB-AN2	-1.10	-1.056	-1.117	-1.091	-1.204	-0.649	-0.942	
2003	" - 2E	-2.25	-2.484	-2.158	-2.158	-1.834	-3.915	-2.853	
2041	" - 3E	-2.36	-2.556	-2.315	-2.419	-1.993	-4.160	-3.021	
2098	" - 3W	-2.48	-2.688	-2.44	-2.563	-2.129	-4.333	-3.180	
2134	" - 2W	-2.54	-2.844	-2.492	-2.56	-2.197	-4.369	-3.265	
2529	" - AS4	-1.70	-1.776	-1.669	-1.72	-1.553	-2.48	-1.991	
2534	" - AS3	-1.182	-1.884	-1.772	-1.834	-1.663	-2.619	-2.118	

1 - EXISTING DISPLACEMENTS
2 - STAGE 1 SOIL REMOVAL
3 - " 1 " " + JACKING LOAD
4 - " 2 " "
5 - " 2 " " + JACKING LOAD
6 - " 3 " "
7 - " 3 " " + JACKING LOAD

ASSUMPTIONS

1 ONLY 13 ELEMENTS REDUCED IN STIFFNESS

Attachment 4

CALCULATED DISPLACEMENTS AT DEEP SEATED BENCHMARKS

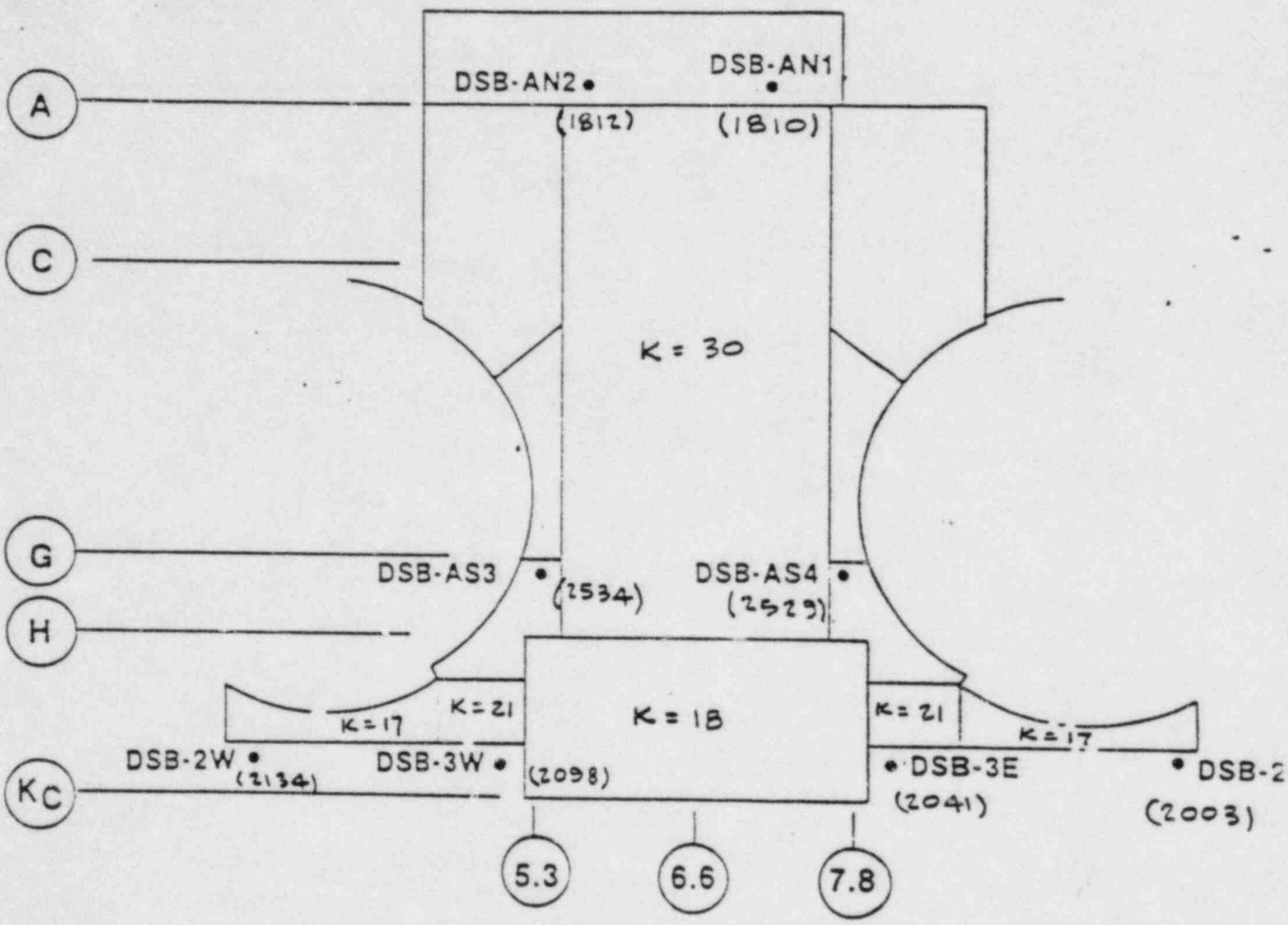


Figure 1

*Exact locations are shown on drawings C-1490 and C-1491

G-25-8-02

Attachment 4

BECHTEL

DISPLACEMENTS WITH K = 30 KCF

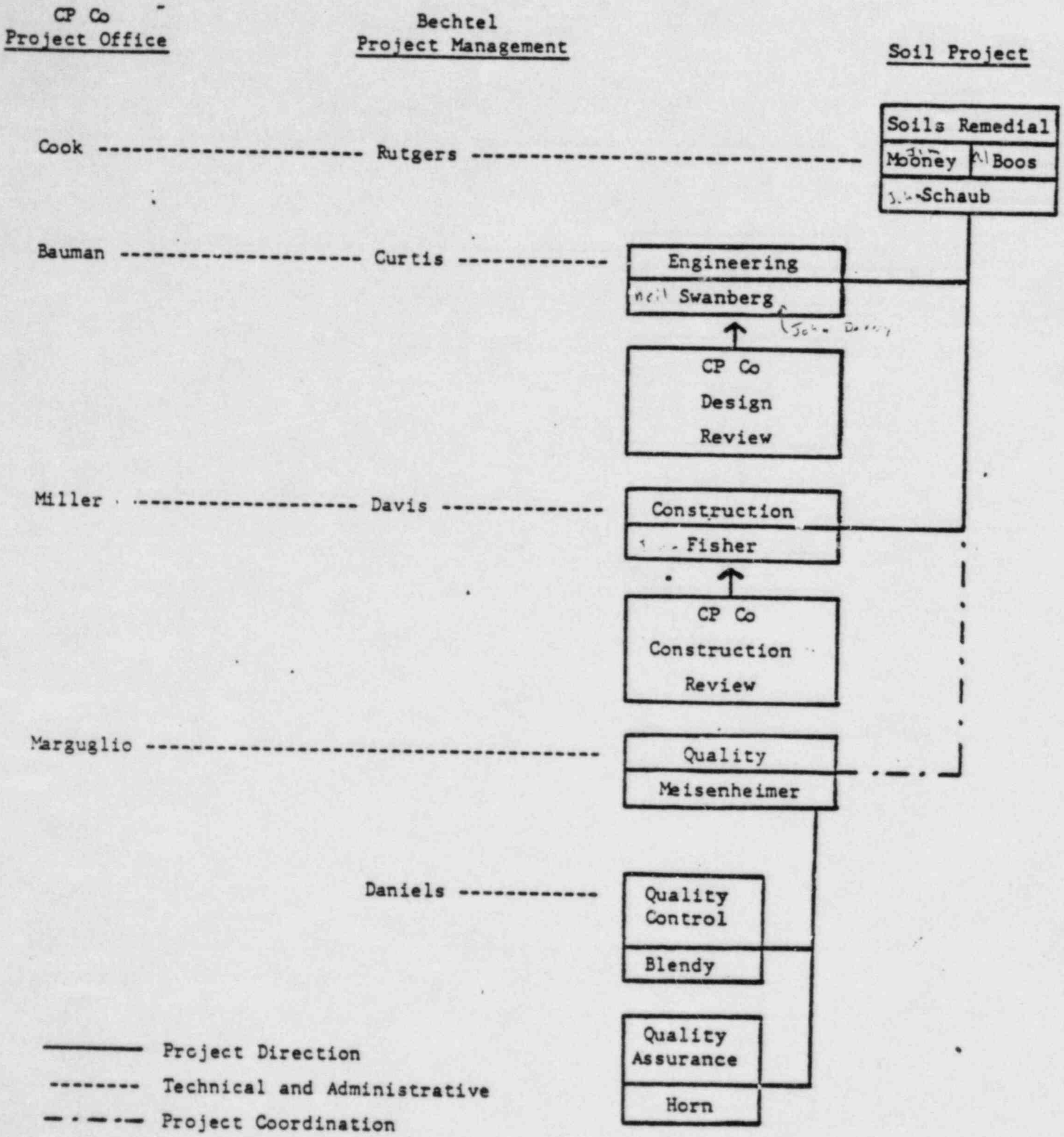
	EXISTING	1ST STAGE		2ND STAGE		3RD STAGE		4TH STAGE	
		3A	3A+3B	4A	4A+4B	5A	5A+5B	7A	7A+7B
1810	-2.79	-2.79"	-2.81"	-2.80"	-2.88"	-2.82	-2.93	-2.77	-2.83
1812	-2.83	-2.851"	-2.87"	-2.86"	-2.95"	-2.88	-2.99	-2.86	-2.92
2003	-3.26	-3.41"	-3.19"	-3.3"	-2.66"	-3.22	-2.38	-3.46	-3.12
2041	-3.32	-3.44"	-3.29"	-3.36"	-2.72"	-3.31	-2.44	-3.65	-3.28
2098	-3.34	-3.516"	-3.37"	-3.46"	-2.80"	-3.41	-2.52	-3.8	-3.42
2134	-3.46	-3.62"	-3.42"	-3.53"	-2.88"	-3.47	-2.62	-3.86	-3.50
2529	-3.08	-3.10	-3.04"	-3.06"	-2.76	-3.06	-2.63	-3.28	-3.08
2534	-3.11"	-3.16	-3.01"	-3.13"	-2.83"	-3.14	-2.69	-3.34	-3.20

3A - STAGE 1 SOIL REMOVAL
3A + 3B - " " " + JACKING LOAD
4A - " 2 " "
4A + 4B - " 2 " " + JACKING "
5A - " 3 " "
5A + 5B - " 3 " " + JACKING "
7A - " 4 " "
7A + 7B - " " . + JACKING "

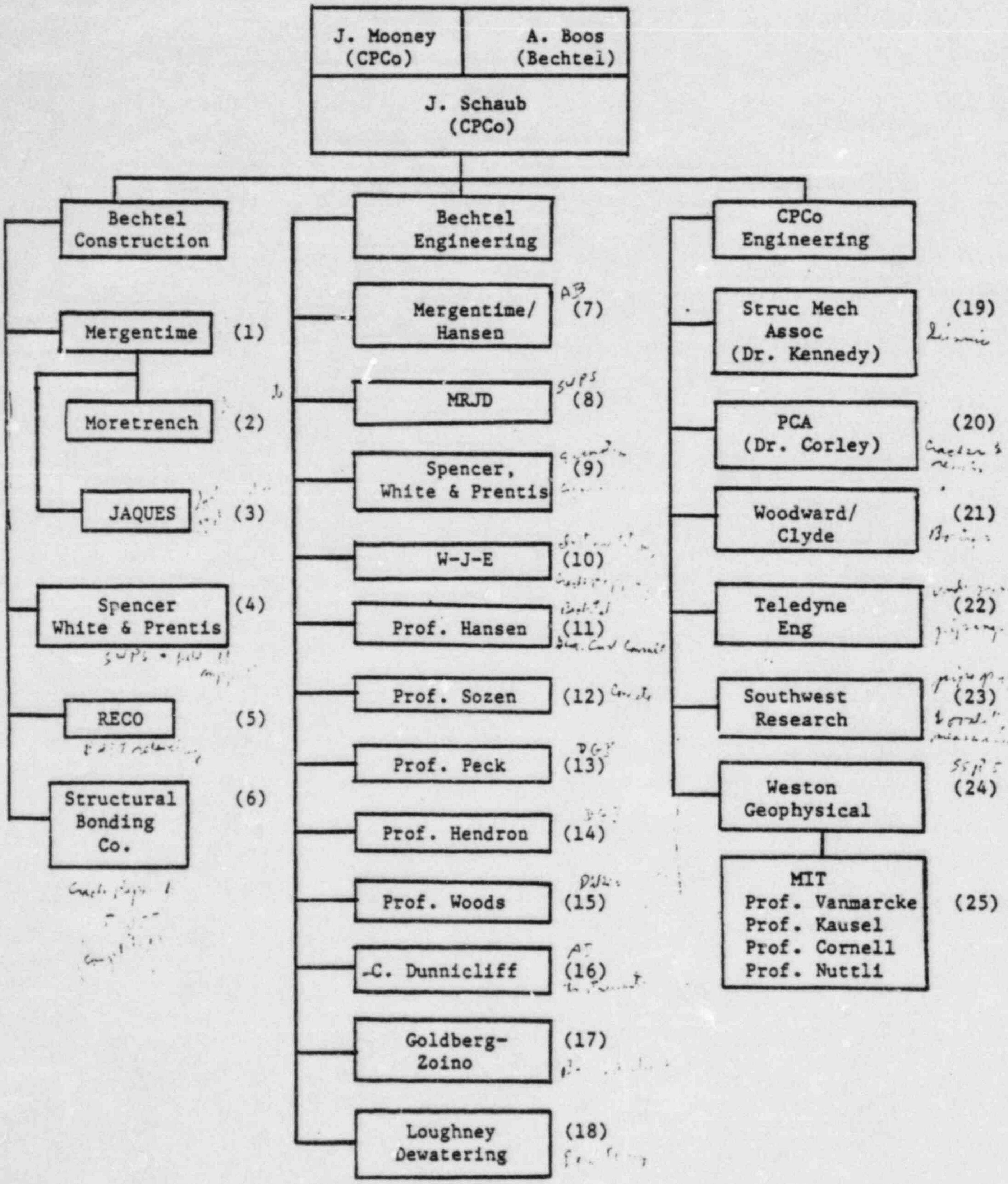
attachment 5

7/29/82

MIDLAND PROJECT



LIST OF SPECIALTY CONSULTANTS
AND SUBCONTRACTORS FOR
MIDLAND REMEDIAL SOILS WORK



LIST OF SPECIALTY CONSULTANTS
AND SUBCONTRACTORS FOR
MIDLAND REMEDIAL SOILS WORK

1.	Subcontractor	Performing underpinning of auxiliary building and FIVP foundation material replacement
2.	Subcontractor	Responsible for groundwater control in support of auxiliary building underpinning
3.	Subcontractor	Responsible for soils stabilization (if necessary)
4.	Subcontractor	Performing service water pump structure underpinning; also providing system for temporary support of utilities during fill replacement north of SWPS and CWIS
5.	Subcontractor	Has developed a proposal for and will relevel borated water storage tank 1T-60
6.	Subcontractor	Performed crack repair on BWST foundations
7.	Consultant	Providing input for design of auxiliary building underpinning and review major underpinning details of auxiliary building
8.	Consultant	Providing input for design of service water pump structure underpinning and review major underpinning details of auxiliary building and SWPS; also providing overview of construction at the Midland jobsite
9.	Consultant	Providing input for integrating SWPS underpinning and removal of soil in designated part of service water piping
10.	Consultant	Providing instrumentation of auxiliary building and SWPS to detect movement and measure strain of selected points; also developed procedures and performed crack mapping in auxiliary building and SWPS
11.	Consultant	Bechtel chief civil engineer's staff; reviews structural model, analytical technique and results of analysis for auxiliary building, SWPS, and BWST
12.	Consultant	Provides input to Bechtel regarding behavior of concrete, including variation of staffness due to cracking in concrete

13.	Consultant	Provided recommendations on remedial action for the diesel generator building and the general approach to permanent plant dewatering and underpinning
14.	Consultant	Provided recommendations on remedial action for the diesel generator building and the general approach to permanent dewatering and underpinning; provided testimonies on static and seismic stability, ECWR dikes, and the BWST soils aspects
15.	Consultant	Made dutch cone and shear wave velocity measurements; performed dike stability calculations and settlement calculations
16.	Consultant	Provided consulting services on instrumentation for diesel generator building
17.	Subcontractor	Performed laboratory and field soil tests and installed and monitored instrumentation
18.	Consultant and Subcontractor	Provided consulting and subcontract service on site temporary dewatering; subcontractor to SW&P on SWPS temporary dewatering
19.	Consultant	Provided overview of design basis, seismic criteria, and dynamic models for seismic analyses; separately performed seismic margin review for site specific response spectra earthquake
20.	Consultant	Performed evaluation of cracks in concrete structures, specifically, auxiliary building, FIVP, SWPS, and DGB under existing conditions, their effects on structural integrity and serviceability; will also be responsible for evaluation of concrete cracks during underpinning
21.	Subcontractor	Performed soil investigation through boring programs and developed laboratory test results
22.	Consultant	Overall consultant on underground piping; developed acceptance criteria for same
23.	Consultant	Performed pipe profile measurements
24.	Consultant	Developed site specific response spectra; performed seismic hazard analysis and soil amplification studies through fill material
25.	Consultants	Provide consulting services to Weston Geophysical for soil amplification, studies, seismic hazard analysis and seismology

SUMMARY OF SOIL CONSTANTS FOR K-KRETE (1)

Attachment 6
sheet 1

	OBE 0.06g	SSE 0.18g 2 (3)	References
Compression wave velocity	10,000 fps	10,000 fps	1,2
Shear wave velocity	5,000 fps	5,000 fps	1,2
Surface wave velocity	4,675 fps	4,675 fps	1,3
Maximum particle velocity (all wave types)	2.43 in/sec	3.64 in/sec	4
Maximum particle acceleration (all wave types)	23.16 in/sec^2	69.48 in/sec^2	3,5
Soil unit weight	130 pcf	130 pcf	
Poisson's ratio	0.25	0.25	
Angle of internal friction	25°	25°	
Coefficient of lateral pressure	0.33	0.33	
Coefficient of friction	[illegible]	[illegible]	
Shear wave velocity (3)			
E max	3,322 fps	3,322 fps	
E min	1,500 fps	1,500 fps	
Ultimate compressive strength	250 psi	250 psi	
Maximum soil strain in/in	(6.17) 10^{-5} in/in	(1.85) 10^{-4}	1

---

(1) K-KRETE is a brand name for a type of low-strength fly ash concrete to be used in place of compacted backfill.

3 (2) The shear modulus and Young's modulus are assumed to remain constant with shear strain.

2 (3) SSE acceleration has been increased by 50% to provide a margin for the site-specific response spectra.

SUMMARY OF SOIL CONSTANTS FOR K-KRETE (Continued) Attachment 6 Sheet 2 2

REFERENCES: 75

1) TPD Design Guide C-2.44, Seismic Analyses of Structures and Equipment for Nuclear Power Plants, Rev 0 79 80

2) Subsurface Investigation and Foundation Soil Report, Vol 2 of 2, Dec 1975, Appendix 2C 83

3) Iqbal, M.A., and Goodling, E.C. Jr., Seismic Design of Buried Piping, 2nd ASCE Specialty Conference on Structural Design of Nuclear Power Plant Facilities, New Orleans, Louisiana, Dec 1975 85 86 87

4) Newmark, N.M., Blume, J.A., and Kapur, K.K., Seismic Design Spectra for Nuclear Power Plants, ASCE, Journal of the Power Division, Nov 1973 90 90

5) Midland Civil Design Criteria, Standard C-501, Rev 11 93

Enclosure 3

Attachment 7

<u>INDEX</u>

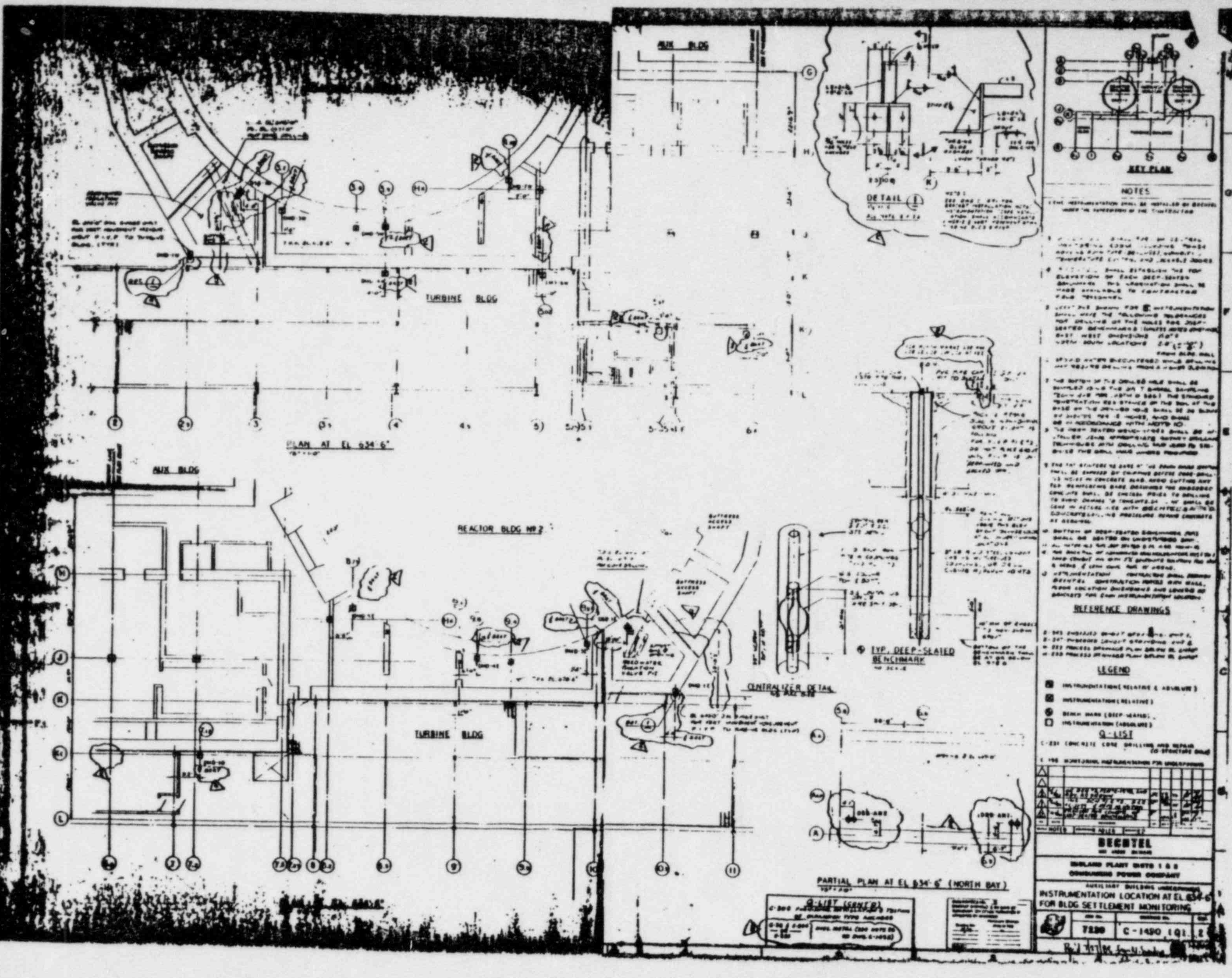
Bechtel Drawing 7220-C-1490(Q), Rev. 2

Bechtel Drawing 7220-C-1491(Q), Rev. 2

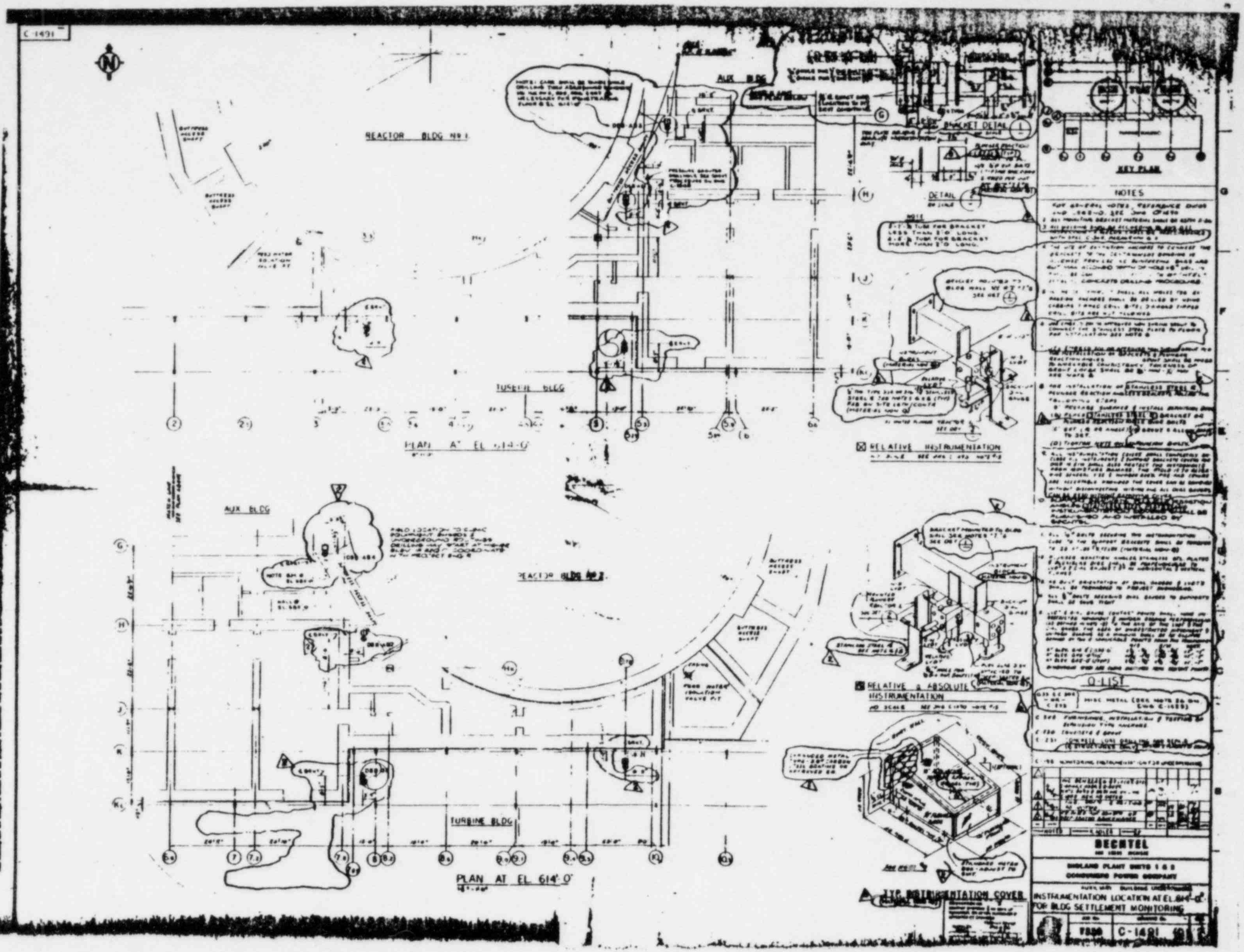
Bechtel Drawing 7220-C-1493(Q), Rev. 1

Bechtel Drawing 7220-C-1495(Q), Rev. 0

OFFICE ▶							
SURNAME ▶							
DATE ▶							



AUX BLDG
TURBINE BLDG
PLAN AT EL 634'-6"
REACTOR BLDG NO 2
KEY PLAN
NOTES
DETAIL
TYP. DEEP-SEATED BENCHMARK
CENTRALIZER DETAIL
REFERENCE DRAWINGS
LEGEND
INSTRUMENTATION (RELATIVE & ABSOLUTE)
INSTRUMENTATION (RELATIVE)
BENCH MARK (DEEP-SEATED)
INSTRUMENTATION (ABSOLUTE)
Q-LIST
PARTIAL PLAN AT EL 634'-6" (NORTH BAY)
Q-LIST (CONT'D)
BECHTEL
MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY
INSTRUMENTATION LOCATION AT EL 634'-6"
FOR BLDG SETTLEMENT MONITORING
7220
C-1490


C-1491
REACTOR BLDG No 1
AUX BLDG
TURBINE BLDG
PLAN AT EL 614'-0"
REACTOR BLDG No 2
PLAN AT EL 614'-0"
KEY PLAN
NOTES
DETAIL
BRACKET DETAIL
RELATIVE INSTRUMENTATION
RELATIVE & ABSOLUTE INSTRUMENTATION
Q-LIST
TYP. INSTRUMENTATION COVER
BECHTEL
MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY
INSTRUMENTATION LOCATION AT EL 614'-0"
FOR BLDG SETTLEMENT MONITORING
7220
C-1491

BUILDING MOVEMENT MONITORING MATRIX

NO.	LOCATION	MONITORED READING	PREOPERATIONAL	FREQUENCY OF READING	INSTRUMENT	ALLOWABLE LIMITS
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

BUILDING MOVEMENT MONITORING MATRIX

NO.	LOCATION	MONITORED READING	PREOPERATIONAL	FREQUENCY OF READING	INSTRUMENT	ALLOWABLE LIMITS
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

NOTES

[illegible]

TABLE 1

△4 MONITORING		
PIER JACKING	MONITORED BENCHMARK	FREQUENCY OF READING
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	
[illegible]	[illegible]	
[illegible]	[illegible]	
[illegible]	[illegible]	
[illegible]	[illegible]	

BUILDING STRAIN MONITORING MATRIX (BEAMS, WALLS AND SLABS)

NUMBER	COLUMN LOCATION	PREOPERATIONAL	MONITORED FREQUENCY OF READING	INSTRUMENTS	PREDICTED ELONGATION	ALLOWABLE ELONGATION
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Q-LIST

CORRECTIVE ACTION DURING UNDERPINNING

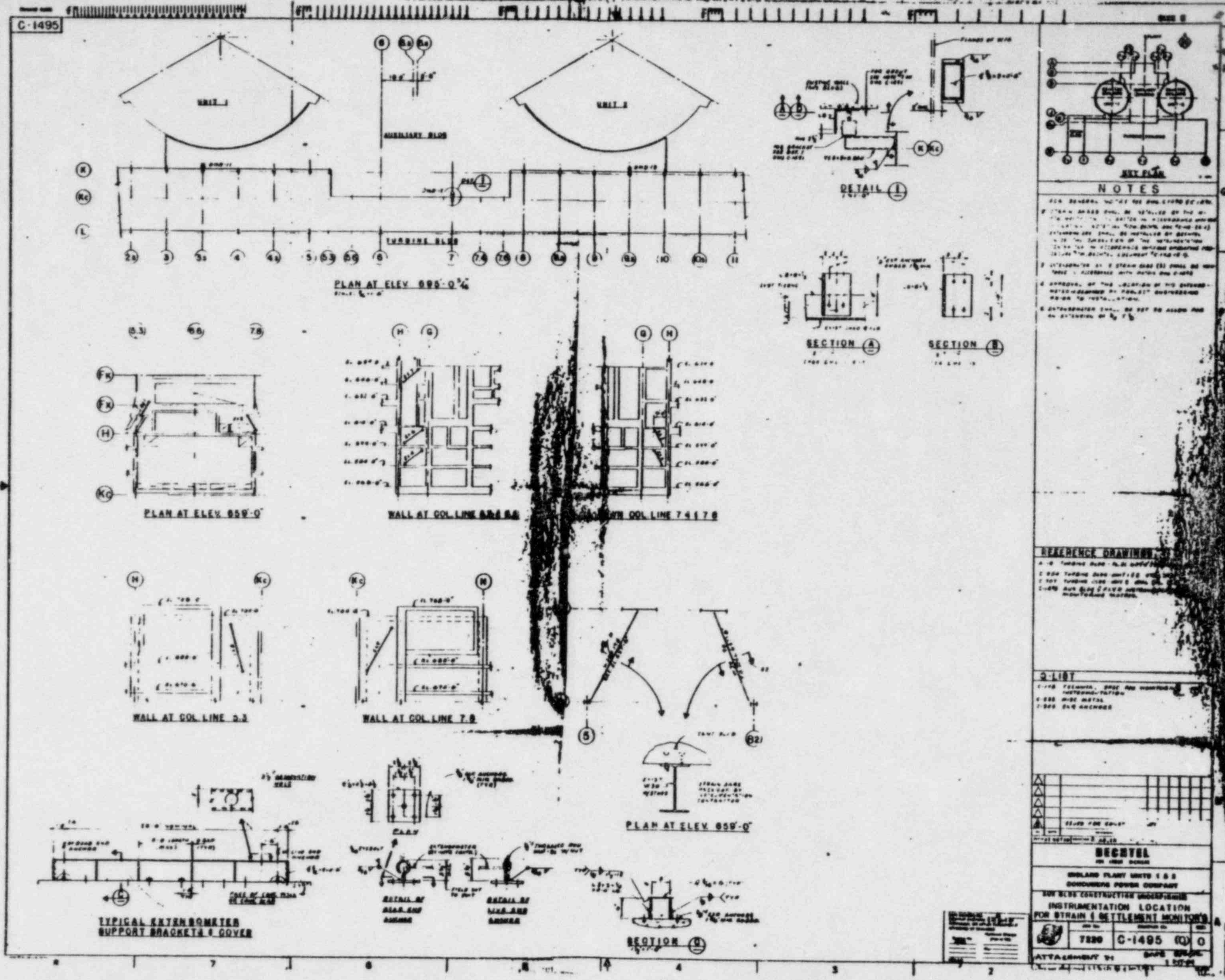
ISSUED FOR CONSTRUCTION

BECHTEL

MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY

AUXILIARY BLDG. & F.I.V.P.
INSTRUMENTATION SYSTEM
MONITORING MATRIX SH. 1

JOB No.	DRAWING No.	REV.
7220	C-1493 (Q)	1



C-1495
UNIT 1
UNIT 2
AUXILIARY BLDG
TURBINE BLDG
PLAN AT ELEV 695'-0
PLAN AT ELEV 659'-0"
WALL AT COL LINE 7.4 & 7.8
WALL AT COL LINE 5.3
WALL AT COL LINE 7.8
PLAN AT ELEV 659'-0"
TYPICAL EXTENSOMETER
SUPPORT BRACKETS & COVER
SECTION
DETAIL
SECTION A
SECTION B
NOTES
KEY PLAN
REFERENCE DRAWINGS
Q-LIST
BECHTEL
MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY
INSTRUMENTATION LOCATION
FOR STRAIN & SETTLEMENT MONITORS
7220
C-1495
0
ATTACHMENT 21