Client: Wisconsin Public Service Company Calculation No. C-020

Title: Fragility Analysis of the Circulating Water Intake and Discharge Piping

Project: Kewaunee USI A-46 / IPEEE

Method: Hand Calculation

Acceptance Criteria: AISC 8th Edition, ACI 318-89, and EPRI NP-6041

Remarks:

REVISIONS

No.	Description	By	Date	Chk.	Date	App.	Date
0	Original Issue	MSL	5/20/94	TMT	5/25/94	WD	5/5/94

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SHEET

FIGURE 1.3

CONTRACT NO.

91C2683

Objective: The objective of this calculation is to compute the fragility (CDFM) value of the Kewaunee Circulating Water Piping (Discharge and Intake) for the IPEEE activity.

In the absence of direct fault displacement or liquefaction, only the following two effects of earthquake ground motion effects on the structures are considered:

A. Axial tension/compression and bending strains induced by the traveling seismic wave.

B. Strain caused by transient horizontal displacement at penetrations.

Conclusion: The CDFM value of the circulating water piping system is 0.418 and it is controlled by the concrete discharge piping at penetration under shear.

References

1. Kewaunee Drawings
   - #S613-G, "Circulating Water Intake & Discharge Plan"
   - #S614-B, "Circulating Water Intake Plan & Profile"
   - #M236-X, "Circulating Water Piping"
   - #XK200-96, "Specification: 120" Reinforced Concrete Pipe with Rubber and Steel Joints (SP-1)"
   - #XK-200-101, "Reinforced Concrete Pressure Pipe with Rubber & Steel Expansion Joint (SP-1) 108" Thru 144" (Floating Rings)"
   - #XK-200-102, "Reinforced Concrete Pipe W/ "S" Stirrups"

2. ASCE Standard 4-86, "Seismic Analysis of Safety Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety-related Nuclear Structures."

3. Iqbal and Goodling, "Seismic Design of Buried Piping", Second ASCE Specialty Conference on Structural Design of Nuclear Plant Facilities, New Orleans, Louisiana, December 8-10, 1975

4. C.K. Wang & C.G. Salmon, "Reinforced Concrete Design, 3rd edition.

5. ACI 318-89 Building Code Requirements for Reinforced Concrete.

6. Boresi, Sidebottom, Seeling & Smith, "Advanced Mechanics of Materials," 3rd edition.

7. Concrete Reinforcing Steel Institute, "Manual of standard Practice," 23rd edition, 1980.

8. EEI Report, "Kewaunee IPEEE," March 1994.

9. Bowles, "Foundation Analysis and Design," 2nd edition.

10. Newmark and Hall, "Development of Criteria for Seismic Review of Selected Nuclear Power Plant," NUREG/CR-0098, May 1978.

11. Kewaunee, "Nuclear Power Plant, USAR".

12. Roark & Young, "Formulas for Stress and Strain," 5th edition.

Assumptions

General

1. Crack developed in the concrete piping is not considered to be failure. It means that leakage is allowed during or after the seismic event.

2. The surcharge due to the soil above the piping structure is ignored in the following calculation.

Discharge Piping

1. Compressive strength of Concrete is 3000 psi.

2. Yield strength of reinforcement is 40 ksi.

3. Reinforcing bars are protected against moisture with sufficient concrete cover.

4. Since the gage 6½ as indicate in Ref. Drwg. # XK-200-96 was not found among the old steel designation gages of Ref. 7. It is assumed that this designation 6-½ should be understood as W6.5. Then diameter of a stirrup is d = 0.288" (Ref. 7 of Page 2-5).

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Intake Piping
1. Yield strength of the steel material is 36 ksi

In the absence of direct fault displacement or liquefaction, the following effects of earthquake ground motion are considered.

a. Axial tension / compression and bending due to traveling seismic wave.

b. Strain caused by transient horizontal displacements at connections.

Parameters:

Soil: Compacted backfill, $\gamma_s = 130$ pcf, $\nu_s = 0.3$ (Ref. 8)

IPEEE Review Level Earthquake (RLE): LLNL ground motion, PGA = 0.306g
max. ground velocity (Ref. 10)
$V_m = 48$ in/sec/g × PGA = 14.7 in/sec

Soil Wave Velocity (Refs. 2, 3 and 8)

Compressional wave velocity, $C_p = 3000$ fps
with $\nu_s = 0.3$ for compacted backfill
Rayleigh wave velocity, $C_R = 0.5 C_p = 1500$ fps
Shear wave velocity, $C_s = C_R / 0.93 = 1613$ fps

Coefficient of subgrade reaction, k_o

From equation 9-14 in Ref. 6, we have

$k_o = 36 q_a$ where q_a = Allowable bearing pressure for soil = 6 ksf (Ref. 11 App. E)
$= 216$ kcf

Coefficient of friction between the pipe and the soil

$f = \pi D \gamma_s h \mu$ Equation 11 of Ref. 3

where D = diameter of the pipe
γ_s = soil density = 130 pcf (Ref. 8)
h = average depth of pipe below grade
μ = coefficient of friction = $\tan(\frac{2}{3}\phi) = \tan\phi = 0.58$
* let $\phi = 1$ (conservative assumption)

$f = 237 Dh$ (pcf)

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Peak Transient Horizontal displacement (Ref. 8)

Following conservative assumptions have been made:

1. The peak transient displacement of the soil deposit due to an earthquake were calculated by integrating with respect to depth the peak values of the seismically induced shear strain in the soil. This is conservative, as the peak shear strains at different depths do not occur simultaneously.

2. The differential displacement between a building and the surrounding ground is conservatively taken to be uniform distributed over a distance of about 25 ft from the foundation.

Single-amplitude transient displacement for PGA of 0.7g, stiff soil profile - Table 3 of EEI report (see page 13)

ground surface 1.2 inches
Screen house 0.5 inches

$$\frac{\text{Screen house disp}}{\text{ground displ.}} = \frac{0.5}{1.2} = 0.42 \leftarrow$$ This ratio is assumed in the following calculation of screen house displacements.

From Fig. 9 of EEI report (see page 12), the single amplitude transient displacement of ground for PGA of 0.45g, stiff soil profile is 0.53 inches.

Using the assumption above, the transient displ. of screen house for PGA of 0.45g stiff soil profile is 0.53 × 0.42 = 0.23 inches.

SRSS is used to compute the differential displacement between the screen house and the ground:

$$\delta = \sqrt{0.53^2 + 0.23^2} = 0.58 \text{ inches}$$

As indicated in Fig. 12 of EEI report, the transient horizontal displacements for very stiff soil profile are much smaller than those for stiff soil profile. Therefore, the displacements for stiff soil profile is used.

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Moment and Shear Capacities of Discharge Pipe

In accordance with Ref. 4 (page 486) the use of gross moment of inertia (i.e. neglecting reinforcement and concrete cracking) underestimates deflections. Hence using it for the purpose of this calculation is conservative.

$$I_g = \frac{\pi(140^4 - 120^4)}{64} = 8.7\text{x}10^6 \text{ in}^4 \qquad E_c = 57000\sqrt{3000} = 3.1\text{x}10^6 \text{ psi}$$

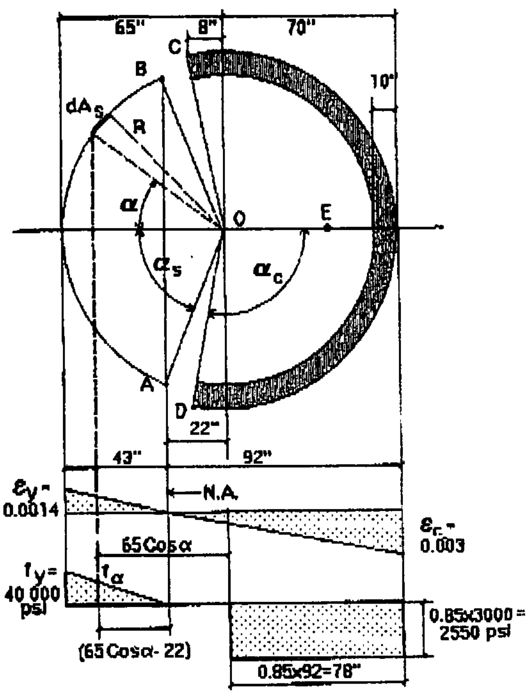
$$A_g = \frac{\pi(140^2 - 120^2)}{4} = 4080 \text{ in}^2$$

$$A_{sh} = \frac{A_g}{k} = \frac{4080}{2} = 2040 \text{ in}^2$$ (k = 2 for a ring from page 173 of Ref. 6)

$$G_c = \frac{3.1\text{x}10^6}{2(1+0.2)} = 1.29\text{x}10^6 \text{ psi}$$ (Poisson ratio = 0.2)

where inside diameter = 10 ft and thickness = 10 in.

<u>Moment capacity</u> of the discharge pipe is calculated similarly to page 410 of Ref. 4:



$$\xi_y = \frac{f_y}{E_s} = \frac{40000}{29\text{x}10^6} = 0.0014$$

So $\alpha_s = 70.22° = 123$ rad. $\alpha_c = 96.56° = 169$ rad.

R = 65 in

$\cup$AB = 65 x 1.23 x 2 = 160 in

$\cup$CD = 70 x 1.69 x 2 = 237 in

CD = 70 x Cos 6.56° x 2 = 139 in

$$0E = \frac{2}{3}\text{x}\frac{70^3 - 60^3}{70^2 - 60^2}\text{x}\frac{139}{237} = 38 \text{ in}$$

Moment due to compression in concrete:

$$A_c = \frac{\pi(140^2 - 120^2)\text{x}2\text{x}96.56}{4\text{x}360} = 2190 \text{ in}^2$$

$$M_c = 0.9 \text{ x } 2550 \text{ x } 2190 \text{ x } 38 = 191 \text{ x } 10^6 \text{ in-lb}$$

Moment due to compression in steel which yields:

$$\alpha = \frac{\pi}{2} - \tan^{-1}\left(\frac{21}{65}\right) = 1.242 rad$$

$$A_s = \frac{3.37}{12} x 2\alpha R = \frac{3.37}{12} x 2x1.242x65 = 45.34 in^2$$

C.O.G. of yielded steel in compression in respect to the cross-section center line (page 69 in Ref. 12):

$$c.o.g = \frac{2R\sin\alpha}{3\alpha}\left(1-\frac{t}{R}+\frac{1}{2-t/R}\right) = \frac{2x70x\sin(1.242)}{3x1.242}\left(1-\frac{10}{70}+\frac{1}{2-10/70}\right) = 49.63 in$$

$$M_T = (22+49.63) x A_s f_y = 71.63x45.34x40000 = 130.x10^6 \; in-lb$$

Moment due to compression in steel which does not yield:

$$A_s = 2x(22+21)x\frac{3.37}{12} = 24.15 in^2$$

$$M_t \approx \frac{2}{3}(22+21) x A_s f_y = \frac{2}{3} x43x24.15x40000 = 27.7x10^6 \; in-lb$$

Moment due to tension in steel:

$$dA_s = \frac{A_s}{\cup AB}R(d\alpha) \qquad f_\alpha = \frac{f_y}{43}(65\cos\alpha - 22)$$

$$\text{Total } A_s = 3.37 \text{ x } (\cup AB) = \frac{3.37x160}{12} = 45 \text{ in}^2$$

$$dT = f_\alpha \text{ x } (dA_s) = \frac{f_y}{43}(65\cos\alpha - 22) \text{ x } \frac{A_s}{\cup AB}R(d\alpha) = \bar{T} \text{ x } (65\cos\alpha - 22) \text{ x } (d\alpha)$$

where $\bar{T}$ is defined as $\bar{T} = \frac{f_y}{43} x \frac{A_s}{\cup AB} x R = \frac{40000x45x65}{43x160} = 17000$ lb/in

$$dM_T = dT \text{ x } 65\cos\alpha = \bar{T}\ (65^2\cos^2\alpha - 1430 \text{ x } \cos\alpha) \text{ x } d\alpha$$

$$M_T = 0.9 \text{ x } 2\int_0^{\alpha s} dM_T = 0.9 \text{ x } 2\bar{T}\left\{65^2\left|\frac{\alpha}{2}+\frac{1}{4}\sin 2\alpha\right|_0^{\alpha s} - 1430\left|\sin\alpha\right|_0^{\alpha s}\right\}$$

$$= 0.9 \text{ x } 2\bar{T}\left[65^2\left(\frac{1.23}{2}+\frac{0.637}{4}\right) - 1430x0.941\right] = 58.9 \text{ x } 10^6 \text{ in-lb}$$

$$\text{Total moment capacity } M = \frac{(191+130+27.7+58.9)x10^6}{10^3 x12} = 33{,}967 \text{ k-ft}$$

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Shear strength capacity is calculated as for 'shear and flexure only' in a beam with a thin-wall circular cross-section with shear reinforcement.

C.O.G. of steel A_S in respect to the cross-section center line:

$$\text{c.o.g.} = \frac{2\int_0^{\alpha_s} dA_s(R\cos\alpha)}{A_s} = \frac{2\int_0^{\alpha_s} \frac{A_s}{\cup AB} R(d\alpha)R\cos\alpha}{A_s} = \frac{2R^2 \sin\alpha_s}{\cup AB} = \frac{2x65^2 \text{Sin}70.22^\circ}{160} = 49.7 \text{ in}$$

d = 49.7 + 70 = 119.7 in

Circumferential reinforcement (two cages with two bar cross-sections):

outside cage $\frac{Av}{s} = 2 \times 1.92 = 3.84 \text{ in}^2/\text{ft}$

inside cage $\frac{Av}{s} = 2 \times 1.44 = 2.88 \text{ in}^2/\text{ft}$

total $\frac{Av}{s} = 6.72 \text{ in}^2/\text{ft}$

From Ref. 9: $V_s = \frac{A_v f_y d}{s} = \frac{6.67x40000x119.7}{12x10^3} = 2680 \text{ kip}$

The flexure-shear cracking load concept has been used similarly to that for rectangular beams (page 121 of Ref. 4 and Section 11.3.2.1 of Ref. 5).

$$\frac{V_u d}{M_u} = \frac{1250xd}{13950} = \frac{1250x119.7}{13950x12} = 0.89$$

$$\text{Total } A_s = 3.37x\frac{130\pi}{12} + \frac{\pi x 0.288^2}{4}60 = 118.6 \text{ in}^2$$

where 0.288 in is stirrup diameter and 60 is total number of stirrups (Drwg. No. XK-200-102).

$$A_s = \frac{\pi(140^2 - 120^2)}{4} = 4080 \text{ in}^2 \qquad \rho = \frac{118.6}{4080} = 0.029$$

$$v_c = 19\sqrt{fc'} + 2500\rho\frac{V_u d}{M_u} \le 3.5\sqrt{fc'}$$

$$v_c = 1.9\sqrt{3000} + 2500 \times 0.029 \times 0.89 = 168 \text{ psi} < 3.5\sqrt{3000} = 190 \text{ psi}$$

Factor k = 1.5 for the ratio of the maximum shear stress to the average shear stress V / A_c (page 173 of Ref. 6) is likely to have been taken into consideration in the formula for v_c above. To adjust this formula for a thin-wall circular cross-section the factor k=2 should be used. The ratio of 2 / 1.5 probably would do the adjustment but since the formula for v_c is based primarily on the experimental information the factor of k = 2 is used here for conservatism.

$$\text{Shear capacity } V_c = \frac{v_c A_c}{2} = \frac{168x4080}{2x10^3} = 343 \text{ kip}$$

Total shear capacity V = 0.85 (2680 + 343) = 2570 kip

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Moment and Shear Capacities of Intake Pipe

Steel pipe with thickness of 7/8"

Moment capacity:
$S = \pi (60 + 0.5 \times 7/8)^2 \times 7/8 = 10040 \text{ in}^3$
Assume $F_y \geq 36$ ksi Then the allowable stress $\geq 1.7 \times 0.6 F_y \approx F_y = 36$ ksi
Moment capacity is evaluated as $M = 36 \times 10040 = 361 \times 10^3$ kip-in = 30000 kip-ft

Shear capacity:
$A = 2 \pi (60 + 0.5 \times 7/8) \times 7/8 = 332.3 \text{ in}^2$
Assume $F_y \geq 36$ ksi Then the allowable stress $\geq 1.7 \times 0.4 F_y = 24.5$ ksi
Shear capacity with shear shape factor of 2 is evaluated as $V = 24.5 \times 332.3 / 2 =$ 4070 kip

Steel pipe with thickness of 5/8"

Moment capacity:
$S = \pi (60 + 0.5 \times 5/8)^2 \times 5/8 = 7142 \text{ in}^3$
Assume $F_y \geq 36$ ksi Then the allowable stress $\geq 1.7 \times 0.6 F_y \approx F_y = 36$ ksi
Moment capacity is evaluated as $M = 36 \times 7142 = 257 \times 10^3$ kip-in = 21400 kip-ft

Shear capacity:
$A = 2 \pi (60 + 0.5 \times 5/8) \times 5/8 = 237 \text{ in}^2$
Assume $F_y \geq 36$ ksi Then the allowable stress $\geq 1.7 \times 0.4 F_y = 24.5$ ksi
Shear capacity with shear shape factor of 2 is evaluated as $V = 24.5 \times 237 / 2 =$ 2900 kip

Stress Calculation Due to Traveling Seismic Wave

(1) Axial strain due to traveling seismic wave

Shear $\epsilon_{max,S} = \frac{V_{max}}{2C_s} = \frac{14.7/12}{2\times1613} = 3.8\times10^{-4}$

Compressional

$\epsilon_{max,P} = \frac{V_{max}}{C_p} = \frac{14.7/12}{3000} = 4.08\times10^{-4}$

Rayleigh

$\epsilon_{max,R} = \frac{V_{max}}{C_R} = \frac{14.7/12}{1500} = 8.16\times10^{-4}$

(2) Bending strain (Curvature) due to traveling seismic wave

Shear $K_{max,S} = \frac{a_{max}}{C_S^2} = \frac{0.306\times32}{1613^2} = 3.76\times10^{-6}\ \frac{1}{ft}$

Compressional

$K_{max,P} = \frac{a_{max}}{(1.6C_p)^2} = \frac{0.306\times32}{(1.6\times3000)^2} = 4.25\times10^{-7}\ \frac{1}{ft}$

Rayleigh

$K_{max,R} = \frac{a_{max}}{C_R^2} = \frac{0.306\times32}{1500^2} = 4.35\times10^{-6}\ \frac{1}{ft}$

(3) Maximum Axial Strain/stress and Bending Moment

Discharge Piping

Axial tension / Compression Strain

$\epsilon_{max} = \max\{\epsilon_{max,S}, \epsilon_{max,P}, \epsilon_{max,R}\} = 8.16\times10^{-4}$ < concrete rupture strain 0.003

Bending Moment

$M = E_c I_g \max\{K_{max,S}, K_{max,P}, K_{max,R}\} = 3.1\times10^3\ ksi \cdot 8.7\times10^6 \cdot 4.35\times10^{-6} \cdot \frac{1}{12} = 9777$ kip-in.

Intake Piping

Axial tension / Compression stress

$f_a = E_s \max\{\epsilon_{max,S}, \epsilon_{max,P}, \epsilon_{max,R}\} = 30\times10^3\ ksi \times 8.16\times10^{-4} = 24.48$ ksi

Bending Moment

$\sigma_b = E_s R \cdot \max\{K_{max,S}, K_{max,P}, K_{max,R}\} = 30\times10^3\ ksi \cdot (60+\frac{7}{8}) \cdot 4.35\times10^{-6} \cdot \frac{1}{12} = 0.66$ ksi

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Stress Calculation Due to Seismically Induced Transient Horizontal Displacement of Penetrations

(1) Axial Movement

Discharge pipe

Tension / compression

$\sigma_c = \frac{P}{A}$ where $P = \sqrt{2 E_c A_c f \delta}$ (Equation 44 in Ref. 3)

$f = 237 D h = 237 \cdot (10' + 20''/12) \cdot 15'$ ← average depth below grade

$= 41.5 \times 10^3$ lbs/ft $= 3.5$ kips/in

$A_c = 4084$ in^2

$\delta = 0.58$ inches for PGA $= 0.45g$

$E_c = 3.1 \times 10^3$ ksi

$\sigma_c = \frac{\sqrt{2 \times 3.1 \times 10^3 \cdot 4084 \cdot 3.5 \cdot 0.58}}{4084} = 1.76$ ksi

and $P/f = 170$ ft < length of pipe = 276 ft.

Intake Pipe ($t = 7/8''$)

Tension / compression

$A_s = 2\pi R t = 2\pi \cdot 5 \times 12 \times 7/8 = 330$ in^2

$f = 237 D \cdot h = 237 \cdot 10 \times 10$ ← average depth below grade

$= 23.7 \times 10^3$ lb/ft

$= 1.98$ kip/in

$\sigma_c = \frac{\sqrt{2 \times 30 \times 10^3 \times 330 \times 1.98 \times 0.58}}{330} = 14.45$ ksi

and $P/f = 220$ ft < length of pipe = 1000 ft

(2) Lateral Movement

Discharge Pipe

$M_d = \frac{k \delta}{2 \lambda^2}$ and $V_d = \frac{k}{\lambda} \delta x$ (Equations 45 & 46 in Ref. 3)

where $k = k_o D = 216$ kcf $\cdot (10' + 20''/12) = 2520$ ksf

$\lambda = \sqrt[4]{\frac{k}{4EI}} = \sqrt[4]{\frac{2520 \text{ ksf}/12^2}{4 \times 3.1 \times 10^3 \text{ ksi} \times 8.7 \times 10^6 \text{ in}^4}} = 0.0036 \frac{1}{in}$

$\delta = 0.58$ inches for PGA $= 0.45g$

$M_d = \frac{2520 \text{ ksf}/144 \times 0.58 \text{ in}}{2 \times (0.0036 \text{ 1/in})^2} = 391600$ kip-in $= 32632$ kip-ft

$V_d = \frac{2520 \text{ ksf}/144 \times 0.58}{0.0036} = 2819$ kips

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Intake Pipe (thickness = 7/8)

$M_d = \frac{k\delta}{2\lambda^2}$ and $V_d = \frac{k}{\lambda}\delta$

where $k = k_0 D = 216 \times 10' = 2160$ ksf

$\lambda = \sqrt[4]{\frac{k}{4EI}} = \sqrt[4]{\frac{2160/144}{4 \times 30 \times 10^3 \times 10040 \times 60}} = 0.0038 \frac{1}{in}$

$M_d = \frac{2160/144 \times 0.58}{2 \times (0.0038)^2} = 301250$ kip-in $= 25100$ kip-ft

$V_d = \frac{2160/144 \times 0.58}{0.0038} = 2290$ kips.

CDFM Analysis

Capacity of Discharge Pipe
Compression / tension: As indicated in Drawing # XK-200-101, the concrete pipe is free to slide several inches at each expansion joint.
Moment : 33967 kip-ft
Shear : 2570 kips

Capacity of Intake pipe
compression / tension : $\sigma_a = F_y = 36$ ksi
Moment : 30000 kip-ft
Shear : 4070 kips.

The controlling section is the discharge concrete pipe at penetration.

Thus, $CDFM = F_A \frac{\sigma_u}{\sigma_{CRE}} S_{CRE}$ where F_A = ductility factor (1.25 if ductile failure, 1.0 if brittle)
σ_u = Ultimate capacity applicable to HCLPF Calculation
σ_{CRE} = Calculated Stress due to IPEEE RLE
S_{CRE} = Max Ground Accel. of IPEEE RLE

Moment: $CDFM = 1.0 \times \frac{33967}{32632} \times 0.45g = 0.47g$

Shear: $CDFM = 1.0 \times \frac{2570}{2819} \times 0.45g = 0.41g$ ← In control

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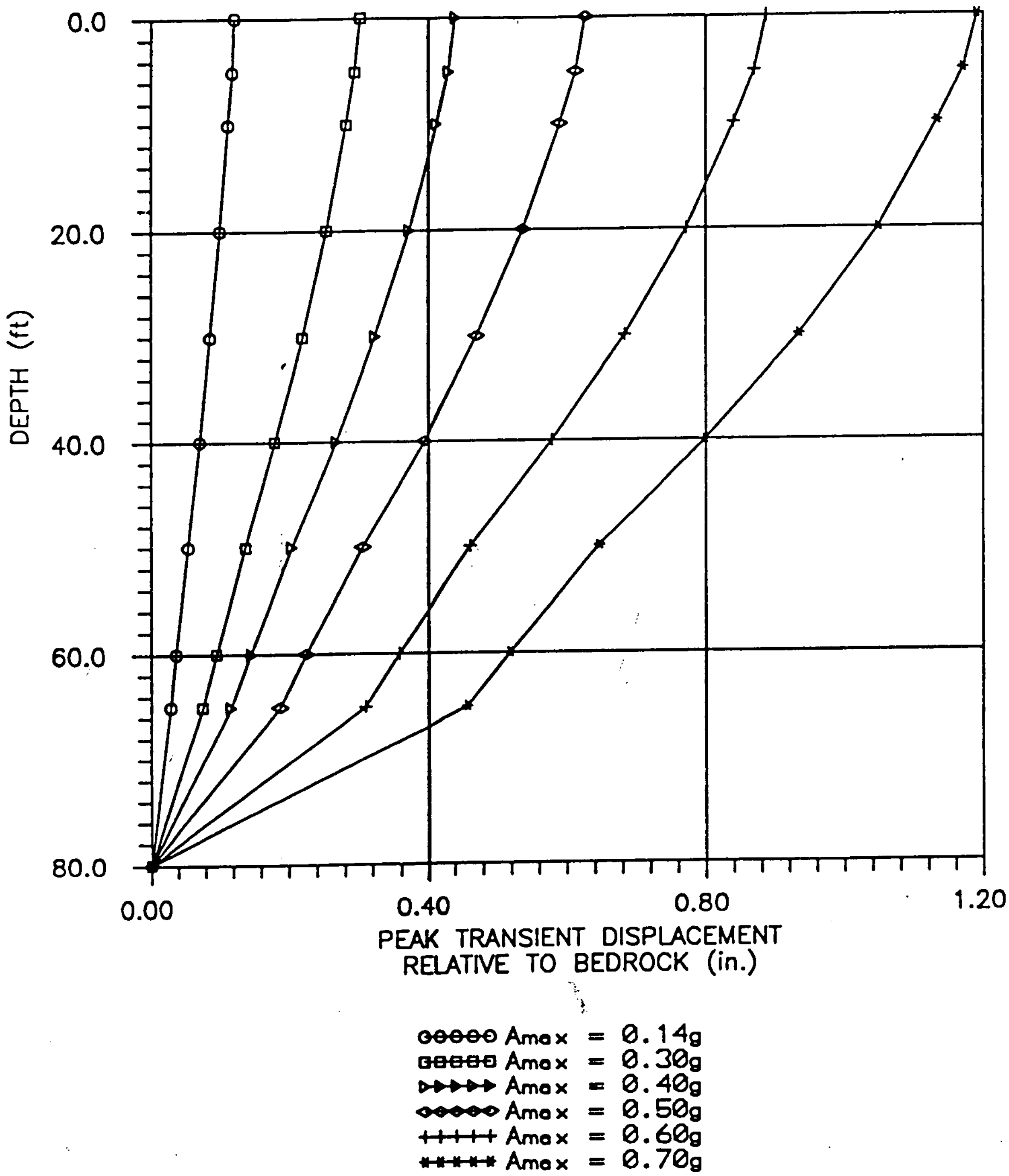

Stevenson & Associates Woburn, Massachusetts	Soil Failure Analysis Kewaunee IPEEE Carlton, Wisconsin	SHAKE RESULTS STIFF PROFILE
Φ GEI Consultants, Inc.	Project 92151	March 1994 Fig. 9

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TABLE 3 - SINGLE-AMPLITUDE AND DIFFERENTIAL TRANSIENT DISPLACEMENTS (INCHES) FOR PEAK GROUND SURFACE ACCELERATION OF 0.7 g, STIFF SOIL PROFILE[1)]
Kewaunee IPEEE
Carlton, Wisconsin

		Differential Displacements (inches)			
	Single-Amplitude Displacement (inches)	Ground Surface	Screenhouse Structure	Turbine and Auxiliary Buildings	Reactor Building
Ground Surface	1.2	0	1.7	2.2	2.1
Screenhouse Structure	0.5	1.7	0	1.5	1.4
Turbine and Auxiliary Buildings	1.0	2.2	1.5	0	1.9
Reactor Building	0.9	2.1	1.4	1.9	0

Note:

1) Results for very stiff soil profile are much smaller.

SCREENING EVALUATION WORK SHEET (SEWS)		GIP Rev 2, Corrected, 2/14/92 Status: Yes Sheet 1 of 2
ID : 153-021 (Rev. 0)	Class : 21 - Tanks and Heat Exchangers	
Description : TANK-REFUELING WATER STORAGE TANK		
Building : AUX	Floor El. : 586.00	Room, Row/Col : 5.0/H.0
Manufacturer, Model, Etc. :		

BASIS : External analysis

1. The buckling capacity of the shell of a large, flat-bottom, vertical tank is equal to or greater than the demand.	Yes
2. The capacity of the anchor bolts and their embedments is equal to or greater than the demand.	Yes
3. The capacity of connections between the anchor bolts and the tank shell is equal to or greater than the demand.	Yes
4. Attached piping has adequate flexibility to accommodate the motion of a large, flat-bottom, vertical tank.	Yes
5. A ring-type foundation is not used to support a large, flat-bottom, vertical tank.	Yes

IS EQUIPMENT SEISMICALLY ADEQUATE? **Yes**

COMMENTS

Tank is braced with 16 lateral braces, 1 per quadrant on approx 20 ft increments.
Anchorage: is 8 approx. 7/8" diameter. Bolt chairs are flat plate welded to gussets on both sides. Some minute cracking noted in base pad.

No hazards for tank and no cracked concrete.

It is a well braced tank. No overturn moment and base shear are created under seismic loading. Thus, base anchorage, bolt chairs and tank shell buckling do not need to be reviewed.

Two items which needed to be reviewed were the integrity of brace strucure under seismic loads and the freeboard clearance vs. slosh height. For tank analysis, see S&A calculation # C-018 Appendix G.

Evaluated by: Date:

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Attachment: Pictures

SCREENING EVALUATION WORK SHEET (SEWS)		GIP Rev 2, Corrected, 2/14/92 Status: Yes Sheet 2 of 2
ID : 153-021 (Rev. 0)	Class : 21 - Tanks and Heat Exchangers	
Description : TANK-REFUELING WATER STORAGE TANK		
Building : AUX	Floor El. : 586.00	Room, Row/Col : 5.0/H.0
Manufacturer, Model, Etc. :		

PICTURES

Figure 1 : 153-021

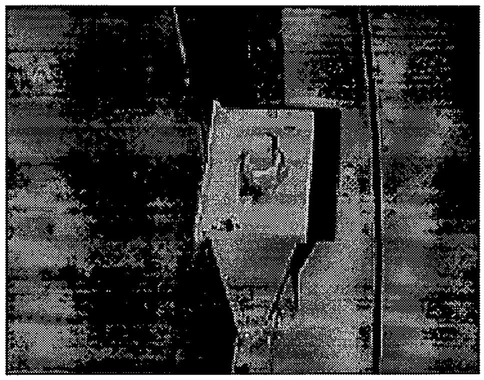

Figure 2 : Anchor for 153-021

Figure 3 : 153-021