Vibration Analysis of the Crystal River Unit 3 Reactor Containment Structure

November 17, 2009
Rev. 2

Summary:

The analysis presented here is part of the root cause investigation by Performance Improvement International of the Reactor Building Containment Wall failure during the Steam Generator Replacement (SGR).

This analysis considers the entire Reactor Building and does not include the detail modeling that may predict local stress concentrations. Further, the wall at the SGR opening location is assumed to be intact in this analysis.

The vibration of the Reactor Building due to the following loads are modeled and analyzed:

- Hydro Cutting of the SGR opening
- Shock Load of Cutting Post-Tensioned Wires

The result of this analysis is:

1. The resonant frequencies of the structure are:
   a. 4.43 Hz – Swaying of Reactor Building
   b. 6.42, 7.48 & 8.37 Hz – Vibration frequencies of Wall Panels between buttresses
2. The modal analysis compares favorably to physical testing (7.4 Hz calculated vs. 7.3 Hz measured).
3. The vibration induced by Hydro Cutting for the SGR opening produces displacements less than 3.5×10^{-4} inch and tensile stresses less than 0.55 psi in the concrete.
4. The shock load of cutting 20 individual Wire Strands of one Hoop Tendon at one time induces vibrations with displacements less than 7×10^{-3} inch and tensile stresses less than 11 psi in the concrete.

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Analysis Objective

The objective of the presented analysis is to study the following two loads for vibrations of the Reactor Containment Structure:

1. Pulsating load of the Hydro Cutting
2. Shock load of cutting of 20 individual Wire Strands of one Hoop Tendon at one time

Modeling Approach and Properties

The geometry of the Crystal River Plant Unit No. 3 Containment structure was modeled based on references 1 though 10. Abaqus version 6.9-1 Finite Element Analysis software was used to model and analyze the structure.

The following components of the structure are included in the model:

1. Concrete base (cylindrical geometry)
2. Concrete wall panels and buttresses (6 panels and 6 buttresses)
3. Concrete dome
4. Interior steel liner (3/8 inch thick wall and roof, 1/4 inch floor)
5. 144 Vertical Tendons (24 ea × 6 Bays) equally spaced around 360 degrees
6. 282 Hoop Tendons (47 ea × 6 Pairs of Bays)

The concrete structure is modeled using the 8-node linear brick elements with incompatible modes, C3D8I, for accurate bending representation. The steel liner is modeled using the 4-node linear shell element S4. The vertical and horizontal tendons are modeled using the 2-node truss element T2D2. The truss elements are embedded in the solid concrete elements with a prescribed initial stress.

The Tendons are made up of 163 individual wire strands, each with a diameter of 7 mm). The Tendons are modeled as being tensioned to 1,400 kip force which results in a prescribed stress of roughly 144,000 lb/in^2.

The Modulus of Elasticity and Mass Density of the concrete is based on Reference 11. The Poisson's Ratio of the concrete is based on Reference 12:

- $E_C = 4.287 \times 10^6$ lb/in^2
- $\nu_C = 0.2$
- $\rho_C = 150$ lb/ft^3

The steel Tendon and Liner properties are taken as:

- $E_S = 30 \times 10^6$ lb/in^2
- $\nu_S = 0.29$
- $\rho_S = 0.282$ lbm/in^3

The bottom of the concrete base is fixed in all translational degrees of freedom.

Finite Element Model Description

The model consists of three types of Finite Elements:

1. Continuum Solid Elements (C3D8I) representing the concrete structure
2. Structural Shell Elements (S4) representing the steel Liner Interior
3. Structural Truss Elements (T2D2) for the representation of the Vertical and Hoop Tendons

Figure 1 depicts the mesh of the concrete structure and the steel liner.

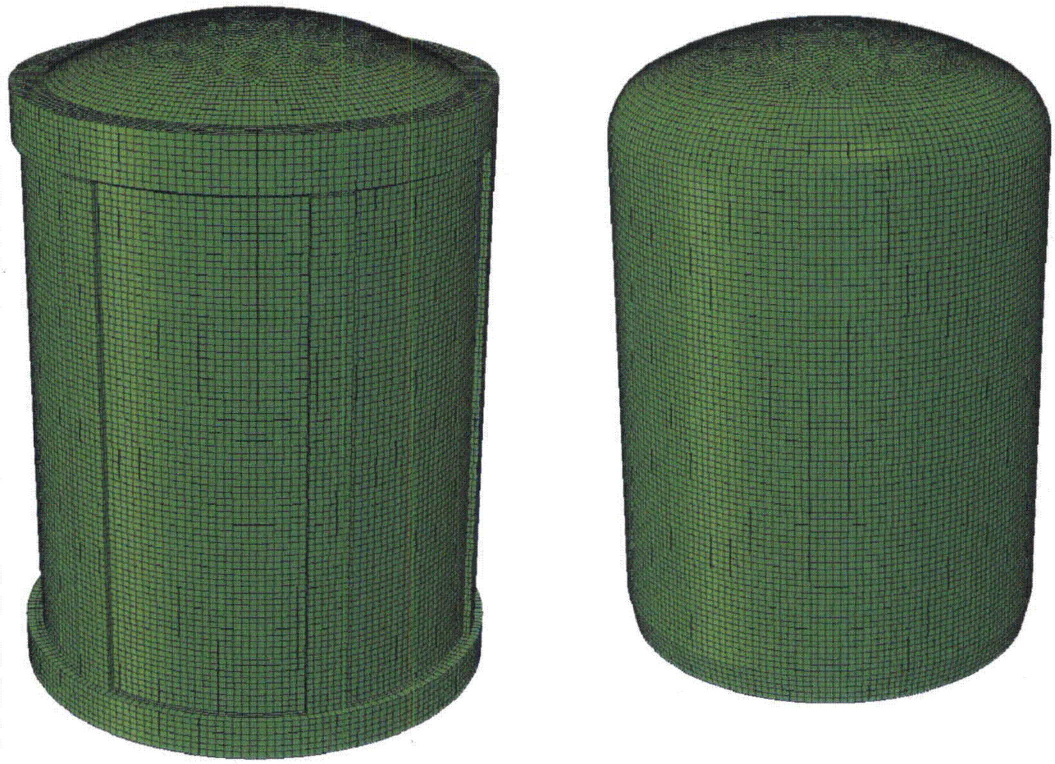


Figure 1: Finite Element Mesh of the Reactor Building. The exterior shown on left and the interior liner mesh shown on the right.

Figure 2 shows the mesh of the Vertical and Hoop Tendons and the location on the solid structure.

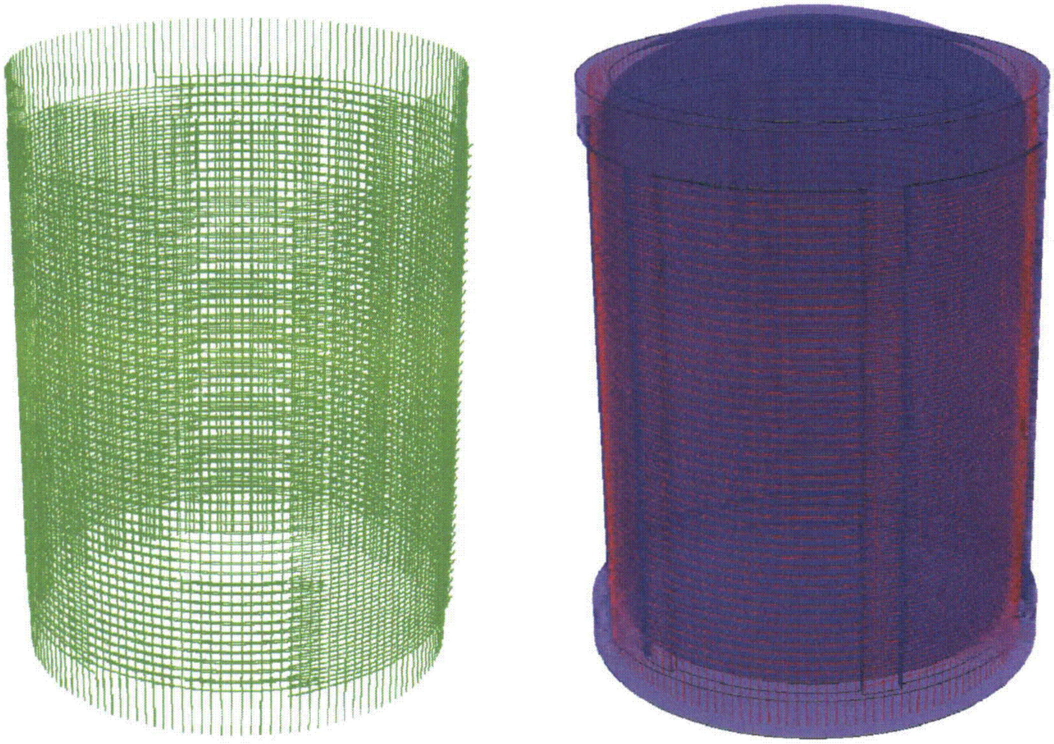


Figure 2: Vertical and Hoop Tendons shown separately on the left and where they are embedded in the concrete solid structure on the right.

Frequency Analysis

The frequency analysis is performed after a static baseline analysis which includes the tensioning of the Tendons and the application of gravity acceleration.

The following vibration modes and associated natural frequencies are found:

- Mode 1 & 2 (4.43 Hz): Swaying of the structure (increasing vibration amplitude with height)
- Mode 3 (6.42 Hz): Radial vibration of the panels of the cylindrical structure. The mode is three panels deflecting in as the other three are deflecting out (every other in and every other out)
- Mode 4 (6.57 Hz): Radial vibration of the cylindrical structure - similar to Mode 3 but rotated 30 degrees such that the buttresses are mostly vibrating instead of the panels
- Mode 5 & 6 (7.48 Hz): Radial vibration of the cylindrical structure with two peaks and two valleys in the circumferential direction
- Mode 7 & 8 (8.37 Hz): Radial vibration of the cylindrical structure with four peaks and four valleys in the circumferential direction
- Mode 9 (8.88 Hz): Twisting of the structure around the vertical centerline.
- Mode 10 & 11 (12.1 Hz): vibration of the cylindrical structure with five peaks and five valleys in the circumferential direction.

Experimental Validation of Frequency Analysis

The natural frequencies of the Containment Structure were measured by Crystal River Unit 3 Personnel (Reference 13). Accelerometers and two types of excitations were employed: (1) hammer impact, and (2) movement of the Polar Crane and lifting equipment during the installation of the Replacement Steam Generator.

The hammer test mostly excites the following frequencies: 7.3 Hz and 15 Hz where the 15 Hz is likely the first harmonic of the 7.3 Hz vibration. However, careful inspection of the spectrum plot of the wall between Buttress 2 and 3 (in Ref. 13) indicates a small peak around 4 Hz. Typically, hammer impact tests may excite more high frequencies than low frequencies. It is likely that the hammer impact test is not exciting the very low frequency associated with the swaying of the structure. In order to excite the tower swaying modes (Mode 1 & 2) a higher amount of energy that a hammer can produce may be needed.

The inspection of the spectrum that was collected during the movement of the crane clearly shows a peak at around 4 Hz. The movement of the crane will likely produce much higher energies and able to excite the bending vibration modes (Mode 1 & 2).

The frequency analysis of the Finite Element Model indicates that the following set of vibration modes exists of the radial vibration of the wall panels (between buttresses):

- Mode 3 (6.42 Hz)
- Mode 5 & 6 (7.48 Hz)
- Mode 7 & 8 (8.37 Hz)
- Mode 10 & 11 (12.1 Hz)

The three closely spaced natural frequencies of the model (6.42, 7.48 and 8.37 Hz) produce the average frequency 7.4 Hz. Depending on what the resolution of the test instrument is and the amount of averaging used, it is likely that the test was unable to resolve the closely spaced frequencies in this range.

It is concluded that the FEA model is likely a good approximation of reality (7.4 Hz calculated vs. 7.3 Hz measured).

Transient Dynamic Analysis of Hydro Demolition

A Modal Dynamic Analysis including the first 50 modes of free vibration along with the natural frequencies and the associated participation factors is utilized to study the response of the Reactor Building due to Hydro Demolition loads. Modal damping equal to 2% of the critical damping for each mode is assumed.

According to Reference 14, the Hydro Demolition equipment has the following characteristics:

- High pressure water is ejected from two heads, each containing three nozzles
- The water is pumped using individual positive displacement pumps operating at 500 rpm (8.33 Hz or 52.36 rad/s)
- The average pressure of the water is 17,000 – 17,500 psi
- The nozzles are 1/8 inch in diameter (0.012272 in^2 opening area)

For this analysis, it is assumed that the pressure pulsation is 25% of the average pressure. It is conservatively assumed that all nozzles are pumping water in phase. The average pressure was converted to a force as follows:

$F_{average}$ = (3 nozzles) × (2 heads) × (17000 psi pressure) × (0.012272 in^2 nozzle opening area) = 1252 lbf

The equation of the time dependent force used in the model is:

(1252 lbf) × (1 + 0.25 × Sin[52.36 *t*]), where *t* is time in seconds.

The force was conservatively concentrated at the most critical point at the midpoint of the bottom SGR opening.

In addition to the abovementioned Hydro Cutting load, the dynamic response to the following loads are also analyzed:

1. Higher pressure (22,000 psi) at 600 rpm
2. Higher pressure (22,000 psi) at 500 rpm
3. Higher pressure (22,000 psi) at 385 rpm (= 6.42 Hz, resonant frequency of Mode 3)

The third additional load listed above is the worst case hypothetical load where the forcing frequency exactly matches the lowest resonant frequency of the wall panels.

Figure 3 shows the maximum Principal Stress of the concrete structure due to load 17,000 psi @ 500 rpm, and 22,000 psi @ 600 rpm.

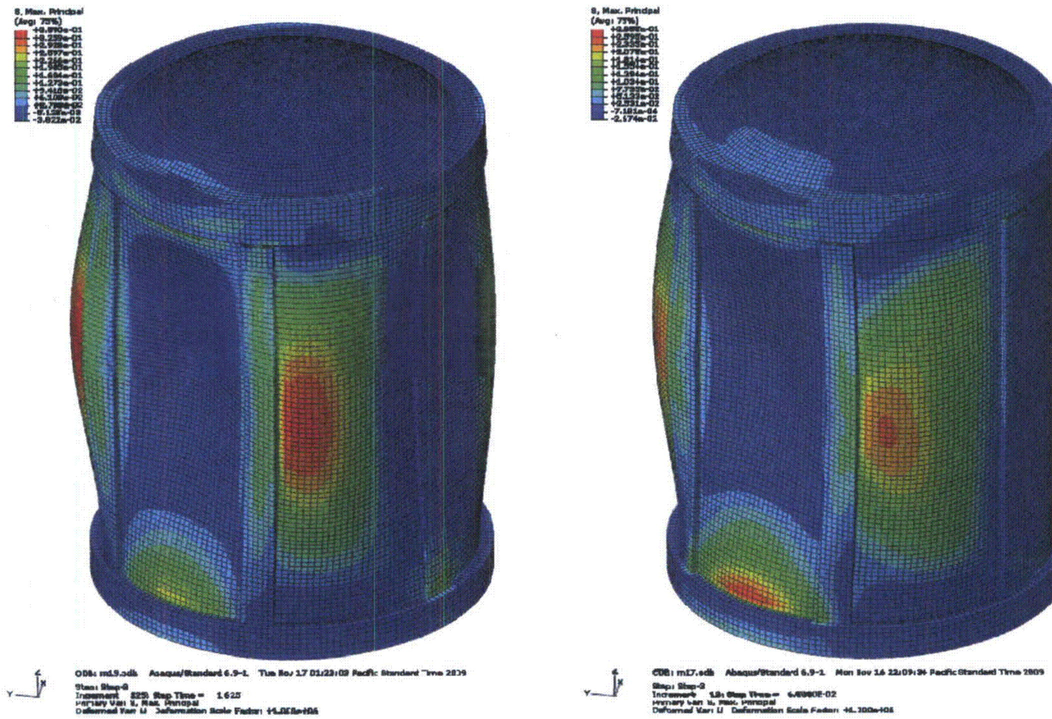


Figure 3: Maximum Principal Stress contours (psi) due to the Hydro Blasting load (17,000 psi @ 500 rpm on the left, and 22,000 psi @ 600 rpm on the right)

Figure 4 shows the maximum displacement of the structure due to the Hydro Blasting loads for the first 2 seconds for loads 17,000 psi @ 500 rpm; 22,000 psi @ 500 rpm; and 22,000 psi @ 600 rpm.

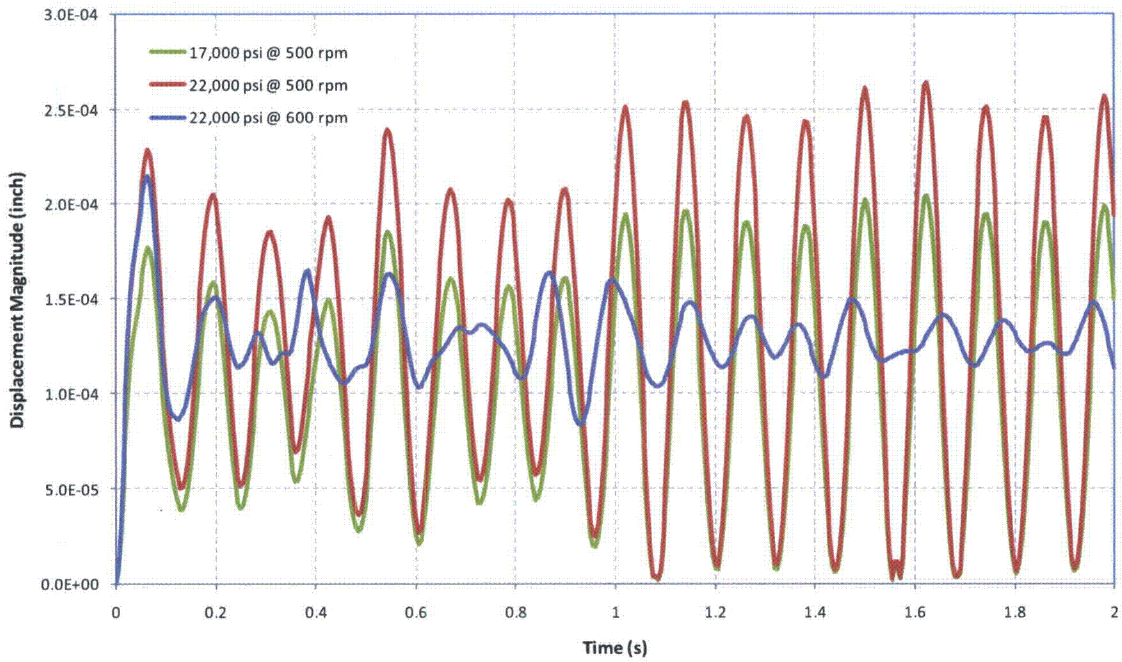


Figure 4: Maximum Displacement Magnitude (inch) of the Reactor Building due to three Hydro Cutting loads for the first two seconds.

Figure 5 shows the dynamic response due to the hypothetical worst case load where the forcing frequency matches the lowest natural frequency of the wall panel. As seen in the figure, the vibration of the wall panel reaches steady state within 4 seconds. After four seconds the amplitude of vibration is not increasing as the damping energy equals the applied energy due to the Hydro Cutting load.

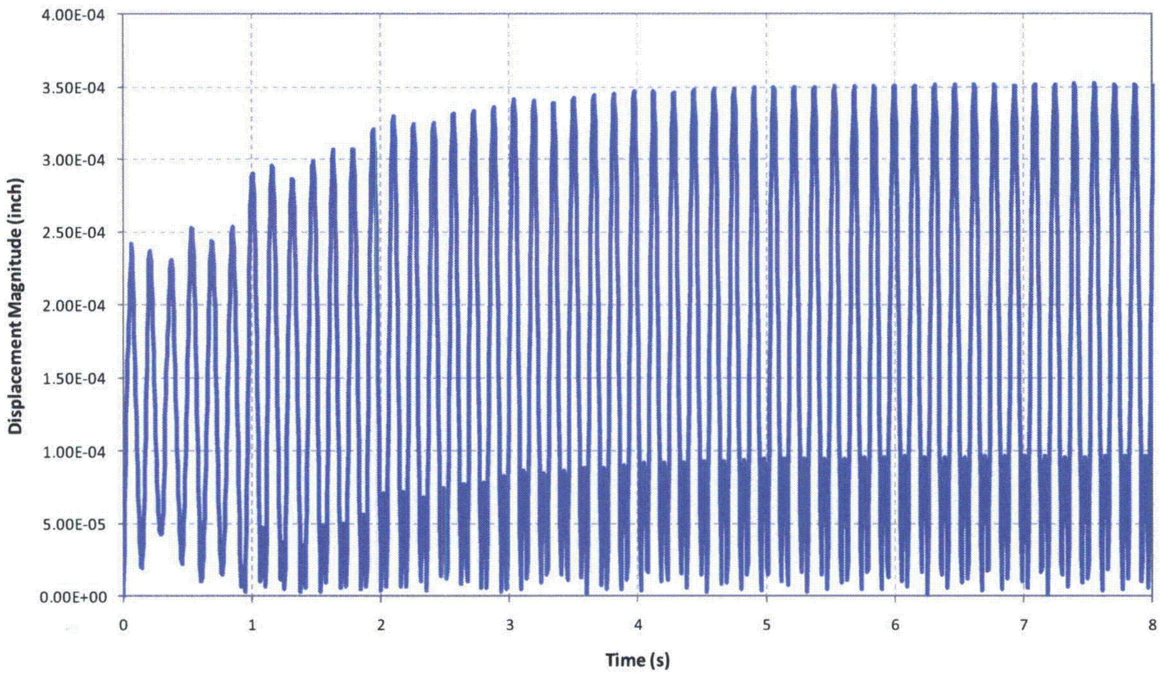


Figure 5: Maximum Displacement Magnitude (inch) of the Reactor Building due to the Hydro Cutting load 22,000 psi @ 385 rpm for the first eight seconds.

The summary of the Maximum Displacement and Maximum Principal Stress due to the Hydro Cutting loads analyzed are listed in Table 1.

Table 1: Displacement and stress results of the concrete structure due to various Hydro Blasting loads.

Hydro Cutting Pressure (psi)	Hydro Cutting Frequency (rpm)	Max Displacement Magnitude (in)	Max Principal Tensile Stress (psi)
17,000	500	2.0×10^{-4}	0.36
22,000	500	2.6×10^{-4}	0.46
22,000	600	2.2×10^{-4}	0.29
22,000	385 (wall resonance)	3.5×10^{-4}	0.55

References

1. Gilbert Associates, Inc., 1970, drawing "Reactor Building Exterior Wall – Concrete Outline"
2. Florida Power, Final Safety Analysis Report, Revision 31.3, Figure 5-2 "Reactor Building Typical Details"
3. Florida Power, Final Safety Analysis Report, Revision 31.3, Figure 5-18 "Base – Cylinder Junction Detail"
4. Florida Power, Final Safety Analysis Report, Revision 31.3, Figure 5-21 "Reactor Building Segments"
5. Florida Power, 1998, drawing "IWE/IWL Inspection Hoop Tendon 13 Layout"
6. Florida Power, 1998, drawing "IWE/IWL Inspection Hoop Tendon 42 Layout"
7. Florida Power, 1998, drawing "IWE/IWL Inspection Hoop Tendon 53 Layout"
8. Florida Power, 1998, drawing "IWE/IWL Inspection Hoop Tendon 64 Layout"
9. Florida Power, 1998, drawing "IWE/IWL Inspection Hoop Tendon 51 Layout"
10. Florida Power, 1998, drawing "IWE/IWL Inspection Hoop Tendon 62 Layout"
11. MPR Associates, Inc., 11/3/2009, "Radial Pressure at Hoop Tendons", Calculation No. 0102-0906-0135
12. Progress Energy, 11/8/1985, "Design Basis Document for the Containment", Rev. 6
13. Memorandum from Virgil W. Gunter, CR3 Systems Engineering, 11/14/2009, "Report of CR3 Containment Structure Vibration Monitoring and Impact Testing"
14. Telephone Interview with Dave McNeill (Mac&Mac, Vancouver), 10/28/2009. Documented in "10 28 interview Dave MacNeil.pdf"

Hydro-Cutter Rotation Effect

The characteristics of the robots used for hydro-demolition of the Reactor Building concrete are described in American Hydro Submittals (to SGT) 1 and 5, both dated 28 Aug 08.

The cutting head on each of the two robots consists of 2 nozzles mounted at opposite ends of a rotating supply pipe. The nozzles are 4.5 inches apart and rotate at a rate that is between 75 and 300 revolutions per minute. Nozzle pressure is 21,000 psi and water jet flow rate (each nozzle) is 75 gallons per minute. American Hydro (AHI) calculations show that jet thrust is 576 pounds force. Since the nozzles are located within just a few inches of the concrete surface, jet force acting on the concrete will be effectively the same as the thrust acting on the nozzles. Jet diameter reported by AHI is 0.157 inches. The thrust and jet diameter reported by AHI are close to the values independently derived for these parameters as shown below.

For a nozzle pressure and flow rate of p = 21,000 psi and f = 75 gpm, respectively:

Velocity, $v = \sqrt{(2\ g\ p / \gamma)}$

where g = acceleration of gravity = 32.2 ft / sec^2
p = pressure = 21,000 psi = 3.0×10^6 lb / ft^2
γ = unit weight of water ~ 62 lb / ft^3

and, $v = \sqrt{(2 \times 32.2 \times 3.0 \times 10^6 / 62)}$ = 1,765 ft / sec

Mass transfer rate, M = f x unit weight per gallon (8 lb / gal) / g

M = 75 x 8 / 32.2 = 18.6 slugs per minute = 0.31 slugs / second

Thrust, F = M x v = 1,765 x 0.31 = 547 lb ~ 576 lb per AHI calculation

(Thrust is rounded to nominal value of 600 lb in the subsequent computations and discussions)

For f = 75 gpm = 289 in^3 / sec and v= 1,765 ft / sec = 21,180 in / sec, area, A_j, of the jet at the vena contracta is:

$A_j = f / v = 289 / 21{,}180 = 0.0136$ in^2

For a coefficient of discharge, $C_d = 0.6$, nozzle area, A_n, is:

$A_n = A_j / C_d = .0136 / 0.6 = 0.0227 \text{ in}^2$

Corresponding nozzle diameter, D_n, is:

$D_n = \sqrt{(4 A_n / \pi)} = \sqrt{(4 \times 0.0227 / \pi)} = 0.17$ inches

The above value computed for D_n is quite close to the 0.157 inch diameter value reported by AHI, who referred to this as the jet diameter but probably meant the diameter of the nozzle.

For a nozzle arm rotation rate of 75 to 300 RPM, every point under the nozzle arc will be subject to a nominal 600 pound load applied at a rate of 150 to 600 times per minute or at a rate of 2.5 to 10 Hz. This force would be sufficient to generate a large amplitude vibratory motion in any small concrete element having a natural frequency in the 2.5 to 10 Hz range. However, small elements of the concrete, such as pieces of coarse aggregate, have a fundamental natural frequency that is far above 10 Hz. A typical coarse aggregate stone, which has a modulus greater than the 4,000,000 psi specified for the concrete as a whole, has a longitudinal wave velocity, $v_w = \sqrt{(E / \rho)}$ where E is the elastic modulus and ρ is density.

For stone with a unit weight of 150 lb / ft^3, density is:

$\rho = 150 / 32.2 = 4.66 \text{ slugs / ft}^3$

For a modulus of 4,000,000 psi = 5.76×10^8 lb / ft^2:

$v_w = \sqrt{(5.76 \times 10^8 / 4.66)} \sim 11{,}000$ ft / sec

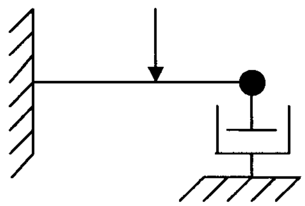
Travel time, t, for an impulse to pass from one face of a ¾ inch stone and reflect back to that face is:

$t = (¾ \times 2 / 12) / v_w = 1.14 \times 10^{-5}$ seconds

The corresponding fundamental natural frequency of the stone for longitudinal waves is 1 / t ~ 88,000 Hz.

Since the fundamental natural frequencies of the individual concrete elements are so far above the greatest (10 Hz) excitation frequency, resonant response to the rotating water jets is not a concern.

Individual Reactor Building structural elements such wall panels between buttresses, have natural frequencies much closer to 10 Hz than to 88,000. While these frequencies could be computed, such a computation is not necessary since a wall panel has dimensions (about 86 ft x 60 ft x 3.5 ft) that are much greater than the 4.5 inch (0.4 ft) diameter of the water jet circle. Because of its size, the wall panel responds to the rotating water jet as it would to a constant point load of 1,200 pound (thrust of both jets on the rotating arm) applied at the center of the jet circle. This conclusion could be demonstrated by a complex finite element calculation. However, it can be qualitatively derived by an analogy that uses a quasi one dimensional (single degree of freedom) model as described below.



The above cantilever assembly consists of a zero mass beam element with a length, L, and a concentrated mass and a dashpot (providing positive, but less than critical, damping) at the free end. The arrow represents a constant downward force moving laterally back and forth from the mass to some point along the length of the beam.

If the force represented by the arrow moves from mass to the point of fixity and back in a time equal to the period of the cantilever assembly, it will excite large amplitude resonant vibrations at the mass end. If it moves from the mass to the mid-point (a distance of L / 2) and back in the same time, it will still excite significant amplitude vibrations. If it moves a distance of L / 4 and back, some level of vibratory motion would still be expected. However, as the distance moved continues to shorten (to L / 8, L / 16 and so on), it will reach a fraction of L at which the motion of the arrow will be imperceptible. At this point, the force can be treated as a fixed load that acts at the end of the beam. As such, it will not generate resonant vibrations in the assembly.

This analogy can be extended to a Reactor Building wall panel. If a 1,200 pound force traversed from the edge to the center of the panel and back in a time equal to the fundamental period of the panel, some level of vibratory response would be expected to result. However, as the distance of travel decreases, the level of response will, as in the example above, be expected to decrease. As the travel distance continues to decrease, it will reach a point at which the resulting vibratory response will be imperceptible.

Using the above discussion a guide, it is possible to intuitively conclude that a travel distance of 4.5 inches (0.4 ft) from the center of a 60 ft wide panel will generate no significant level of oscillatory response in the panel.

The above conclusion is also valid for the Reactor Building as a whole since overall building dimensions are significantly greater than those of a wall panel.

Therefore, in view of the above calculations and analogy, it appears reasonably clear that the rotating hydro-demolition jets will not result in meaningful oscillatory movement of the Reactor Building or its constituent structural elements.

Also, for major structural elements or the building as a whole, resonant responses of interest are generally due to bending. A single application of a force at point on a wall panel induces a bending deformation that results from the product of force and moment arm. If the force is applied and released, the wall panel will cycle in various modes (at various resonant frequencies) with amplitudes decaying due to internal damping. If the force is re-applied at a repetition rate equal to a resonant frequency of the panel, the amplitude of the cyclic motion will increase. The following sequence addresses this amplitude multiplication for a repetitive force applied at the free end of the cantilever beam shown above. A Reactor Building wall panel behaves in a similar, but more complex, manner.

- When the force is applied, the end of the beam deflects by an amount δ determined by the product of the force and the moment arm (L).
- When the force is suddenly released, the beam will oscillate at its natural frequency and, in one cycle, the end will return to a deflection of $k\ \delta$ where k is a factor less than one determined by the damping of the dashpot. At this point in the cycle there are neither external nor inertial forces acting on the beam.
- If the force is reapplied after one cycle, the bending moment generated by the force will increase the deflection of the beam end by an additional amount δ for a total deflection of $(1 + k)\ \delta$.
- After one more cycle, deflection will be $k\ (1+ k)\ \delta$.
- A third application of the force and corresponding moment at this point in time increases the deflection by an additional amount δ for a total deflection of $k\ (1 + k)\ \delta + \delta = (k^2 + k + 1)\ \delta$.
- Following multiple (n) applications of the force at the same periodic interval, the total deflection will be:

$$\text{Deflection} = (k^{n-1} + k^{n-2} + \ldots + k^2 + k + 1)\ \delta$$

As shown above, total deflection continues to increase with each application of the force but deflection is, in fact, determined by moment rather than force; if the force is applied at the fixed end of the beam, there is no resulting deflection at the free end. If the force is only moved through a small distance rather than being applied and released, the following happens.

- On initial application of the force, the free end of the beam still deflects by an amount δ.
- When the force is moved (assume instantly for this example) a small distance to the left, the end of the beam will oscillate but only through an amplitude determined by the difference between the deflection, δ, with the force at the end of the beam and the deflection, δ_1, resulting from the bending produced by the force and the slightly reduced moment arm.
- Deflection at the end of one cycle is $\delta_1 + k(\delta - \delta_1)$.
- Moving the force (again instantly) back to the end of the beam at the end of one cycle increases deflection by an amount, $\delta - \delta_1$, that is determined by the increase in moment arm and moment. Total deflection is then:

  $$\delta_1 + k(\delta - \delta_1) + (\delta - \delta_1)$$

- As developed above, the total deflection at the end of multiple (n) cycles of moving the point of force application is:

  $$\text{Deflection} = \delta_1 + (k^{n-1} + k^{n-2} + \ldots + k^2 + k + 1)(\delta - \delta_1)$$

In the above expression, δ_1 is the fixed displacement resulting from the bending moment due to positioning the force to the left of the end and,

$$(k^{n-1} + k^{n-2} + \ldots + k^2 + k + 1)(\delta - \delta_1) \quad (1)$$

is the amplitude of the cyclic movements after n cycles of moving the force back and forth along the beam. For values of $k < 1$ (positive damping) and for large values of n,

$$k^{n-1} + k^{n-2} + \ldots + k^2 + k + 1 \sim 1/(1 - k)$$

As in any damped vibration with a continuous forcing function, the amplitude has an asymptotic limit determined by the degree of damping. But, more significantly in the current example, the amplitude is also limited by the change in moment, which is determined by the distance through which the force moves along the length of the beam. If the movement is small relative to L, the factor $(\delta - \delta_1)$ in

Expression (1) will be small and the oscillatory amplitude through which the end of the beam moves will be limited to a correspondingly small value.

In the case of the rotating jets on the Reactor Building wall panel, relatively large amplitude vibrations could result if the jet force started and stopped at a frequency close to a natural frequency of the panel. This would be equivalent to moving the jet quickly from the center of the panel to the edge and the quickly back to the center at the same frequency. Moving the jet over the full half width of the panel (about 30 ft) maximizes the change in moment arm and results in maximum oscillatory amplitude. If the jet is moved through a distance of only 4.5 inches (~0.4 ft or about 1% of the panel half width) the change in moment arm and corresponding change in bending moment, as well as resulting oscillatory amplitude, will be small.

The following numerical example provides a conservative order of magnitude estimate of Reactor Building wall panel oscillation amplitude under a jet that rotates at the fundamental natural frequency of the panel.

The curved panel above the equipment opening is approximated as a 3.5 ft (d) 12 ft (b) wide beam spanning 60 ft (L) between buttresses, which are treated as simple supports.

Deflection, δ, under a 1,200 lb line load at the center of the beam is, for a modulus of 4,000,000 psi:

$$\delta = F\,L^3 / (48\,E\,I)$$

$$I = bd^3 / 12 = 12 \times 12 \times (3.5 \times 12)^3 / 12 = 889{,}000 \text{ in}^4$$

$$\delta = 1{,}200 \times (60 \times 12)^3 / (48 \times 4 \times 10^6 \times 889{,}000) = 2.6 \times 10^{-3} \text{ inches}$$

For a small shift, dL, in the position of the load, the change in deflection, $\delta - \delta_1$, will be approximately:

$$\delta - \delta_1 = [dL / (L / 2)]\,\delta$$

For a 0.4 ft shift in the position of the 1,200 lb force:

$$\delta - \delta_1 = (0.4 / 30) \times 2.6 \times 10^{-3} = 0.035 \times 10^{-3} \text{ inches}$$

Assigning a value of 0.95 to k results in the following oscillatory amplitude, A, at the center of the beam.

$$A = 1 / (1 - k) \times (\delta - \delta_1) = [1 / (1 - 0.95)] \times 0.035 \times 10^{-3} = 0.7 \times 10^{-3} \text{ inches}$$

The oscillatory amplitude of the idealized beam is very small; i.e., only about ¼ of the static deflection under the 1,200 lb jet load. The amplitude of the actual reactor building wall panel oscillations will be even smaller than this for at least the following reasons.

- The curved wall is stiffer than the idealized flat beam.
- The idealized beam is 12 ft wide. The curved wall extends from the top of the equipment opening to the ring girder, a distance of about 86 ft, which increases the stiffness.
- The current modulus of the concrete is probably much greater than the 4,000 ksi design value, which also increases the stiffness.
- The natural frequency of the wall is unlikely to match the frequency of jet rotation.
- The rotating jets represent a much less severe oscillatory loading condition than a single 1,200 pound force that is quickly shifted laterally.
- The mass of the concrete is neglected in the above computation. In reality, this inertia of this mass would limit the deflection under the short duration load to less than the amount that was computed considering the stiffness alone.

Finally, it is concluded on the basis of the above computations and discussions that hydro-demolition jet induced vibrations of the Reactor Building wall as well as vibrations of the building as a whole will be negligible and need not be further evaluated.