American Hydro Calculation

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

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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

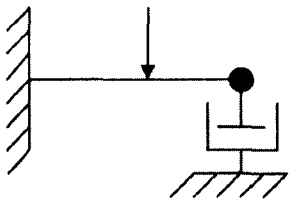
Travel time, t, for an impulse to pass from one face of a ¾ inch stone and reflect back to that face is:

$t = (\frac{3}{4} \times 2 / 12) / v_w = 1.14 \times 10^{-5}$ seconds

The corresponding fundamental natural frequency of the stone for longitudinal waves is $1 / t \sim 88{,}000$ 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,

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

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.

Date: November 14, 2009

To: Charles Williams
CR3 Containment Structure Root Cause Team

From: Virgil W. Gunter
CR3 Systems Engineering

Subject: IOC SE09-0057
Report of CR3 Containment Structure Vibration Monitoring and Impact Testing

CR3 PdM personnel were requested to attempt to determine natural vibration frequencies of the CR3 containment structure. Below are the results of vibration monitoring and impact testing of the CR3 containment structure. This task was attempted utilizing the following methods. Five PCB model 393B31 seismic accelerometers were mounted to the exterior wall of the containment structure at various locations. Vibration data was then collected while striking the structure with a PCB Model 086C42 12lb impact hammer. In addition data was collected at 3 locations using a multi-channel vibration instrument while the "A" Replacement Steam Generator was being moved into and placed in the reactor building. The expectation was the mass of the Replacement Steam Generator being moved by the polar crane and\or the Mammoet lifting equipment would result in impacts that may be captured by the vibration instrumentation.

Vibration data collected utilizing the impact hammer is consistent between all five locations tested. The predominant frequencies identified were 7.3 Hz and 14.96 Hz respectively. The amplitude at 14.96 Hz was generally slightly higher at each of the locations relative to the 7.3 Hz peak when utilizing the impact hammer. Data collected during movement of the Replacement Steam Generator indicated higher amplitudes at the 7.3 Hz peak.

Vibration data collected utilizing the multi-channel vibration instrument, during the movement of the Replacement Steam Generator were generally in agreement with data collected during the impact testing. Below are representative samples of vibration data collected.

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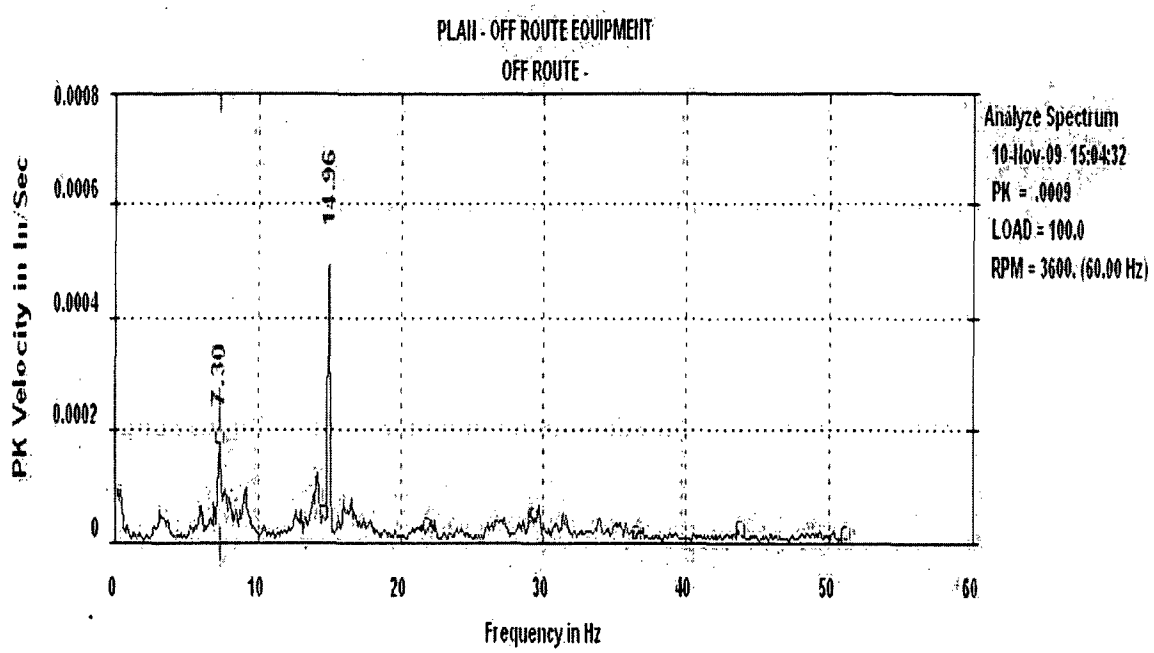
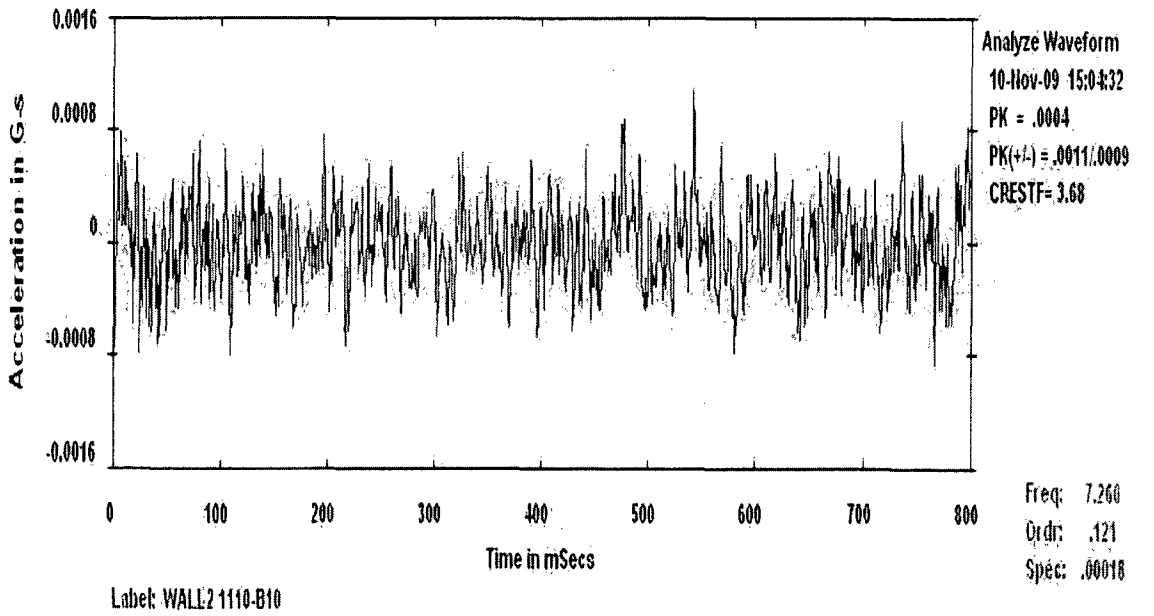
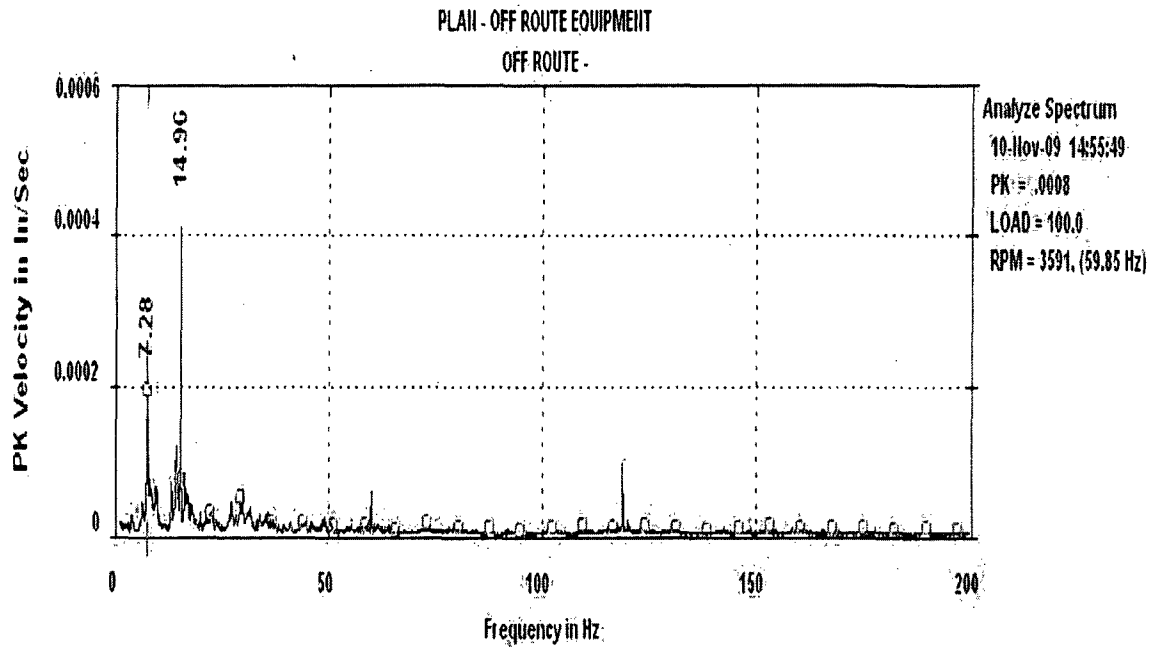
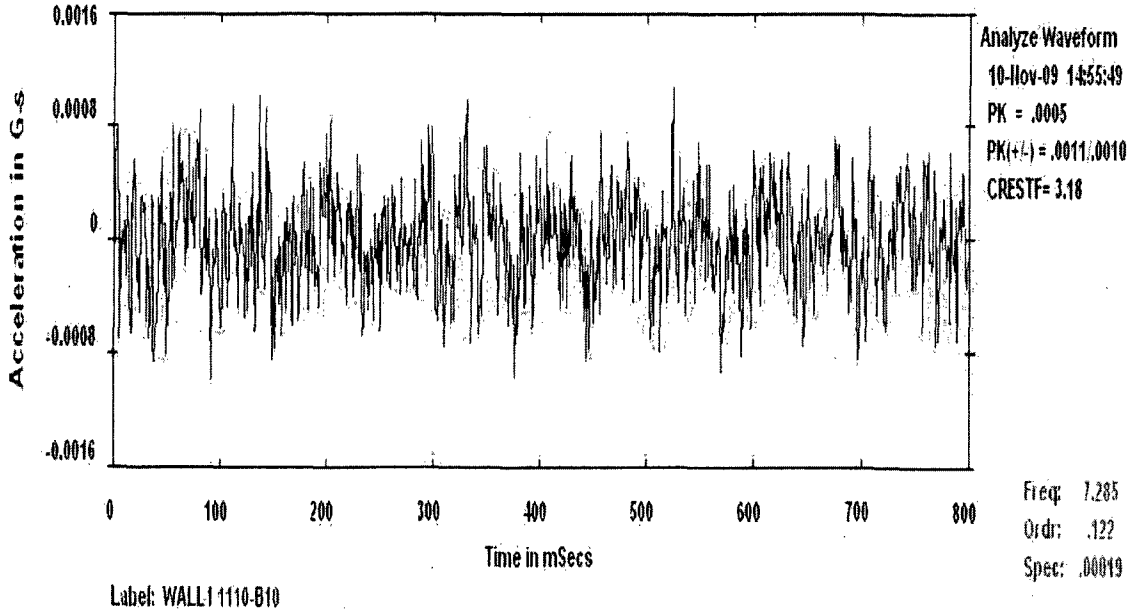
Memorandum Page 2 February 5, 2010



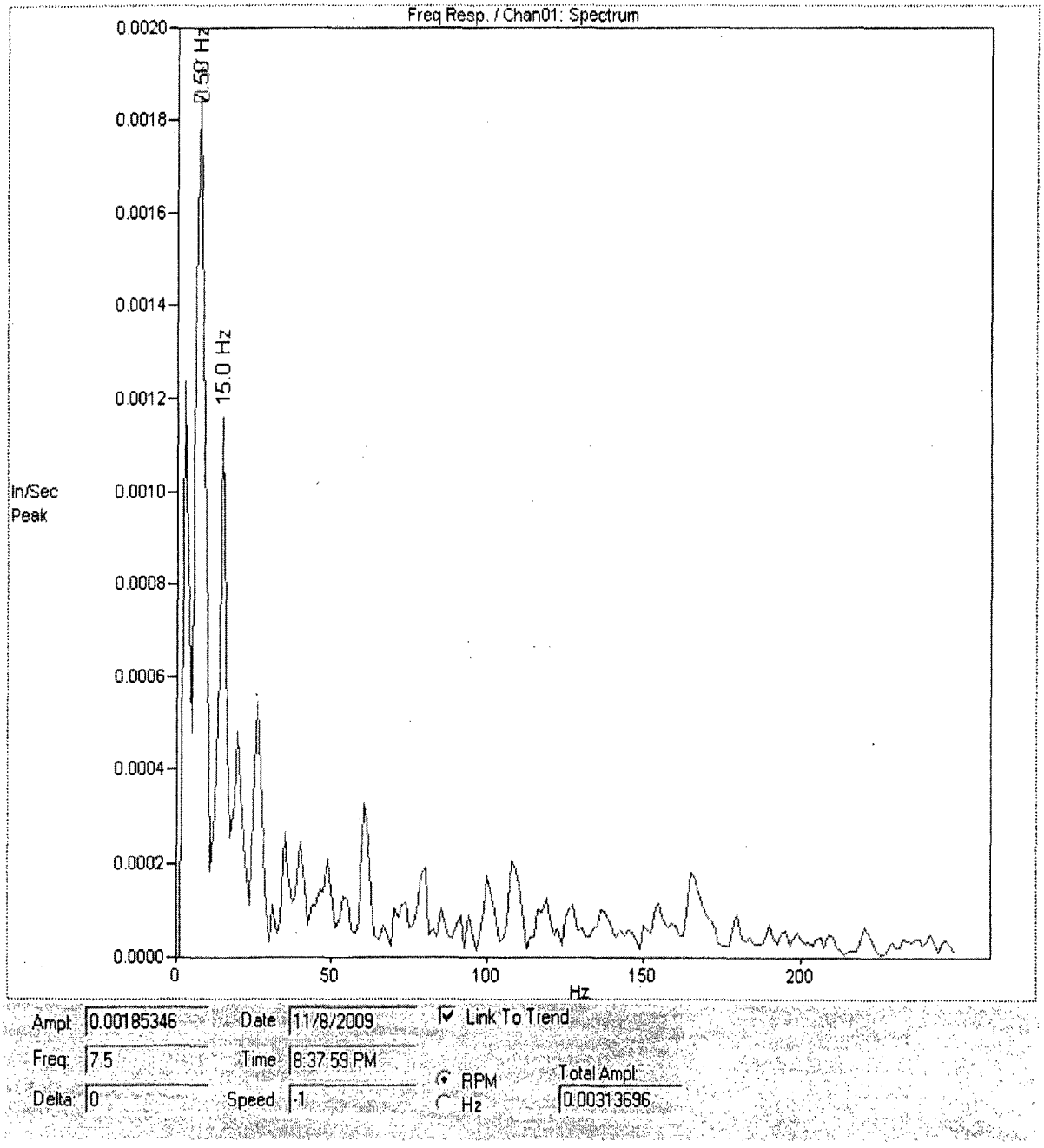


Spectrum and associated Time Wave form collected during impact testing
Wall between buttresses 2 and 3 @ Approximately 90 Deg. Approximately Elev. 155'

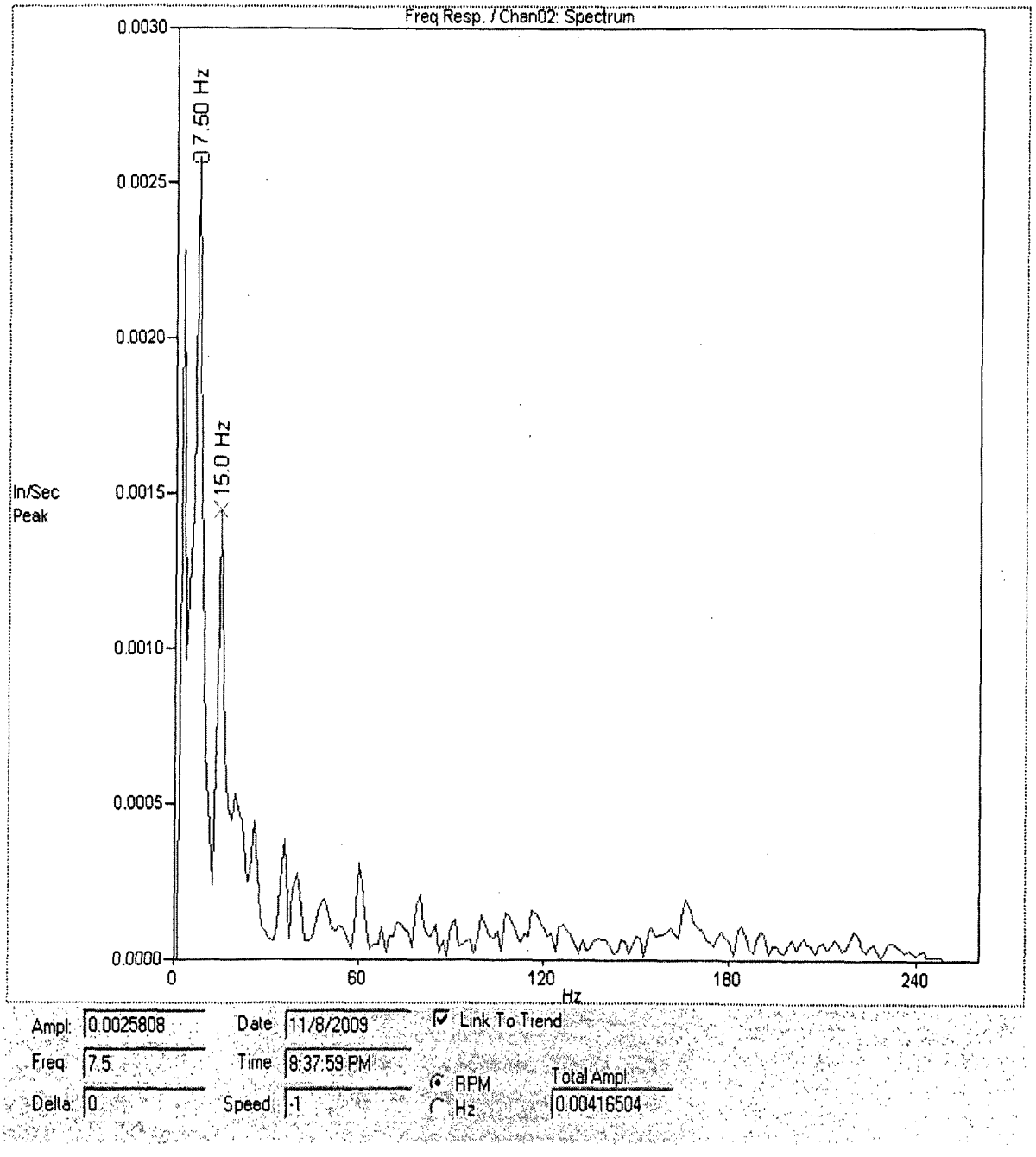




Spectrum and associated Time Wave form collected during impact testing
Wall between Buttresses 1 and 6 @ approximately 350 deg. Approximately Elev. 150'



Spectral Data collected during movement of Replacement Steam Generator
Wall between buttresses 2 and 3 @ Approximately 90 Deg. Approximately Elev. 155'



Spectral Data collected during movement of Replacement Steam Generator between Buttresses 1 and 6 @ approximately 350 deg. Approximately Elev. 150

Accelerometer Locations

Wall between Buttresses 1 and 6 @ approximately 350 deg. Approximately Elev. 150'
Wall between buttresses 2 and 3 @ approximately 90 deg. Approximately Elev. 155' (3 positions)
Wall between buttresses 6 and 5 @ approximately 270 deg. Approximately Elev. 206'

Test Equipment utilized

Seismic Accelerometers Model 393B31
UTC-000 1780492 Cal Due 3-23-2010
UTC- 0001780493 Cal Due 11-6-2010
UTC- 0001780494 Cal Due 1-26-2010
UTC- 0001780495 Cal Due 1-26-2010
UTC- 0001780496 Cal Due 3-23-2010
UTC- 0001780497 Cal Due 3-23-2010

Vibration Instrument Dual Channel
UTC- 0001256706 Cal Due 2-25-2010

Virgil W. Gunter
Sr. Engineer Technical Support Specialist
Crystal River Nuclear Plant

Rich Wieman
Superintendent Systems Engineering
Crystal River Nuclear Plant

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5.7 REFERENCES 90

This report identifies and documents FPC's position on the various issues pertaining to pipe rupture requirements outside containment. The position has been established considering the technical and regulatory requirements at the time of plant design and construction, and current (including Generic Letter 87-11) NRC Standard Review Plan (SRP) guidance, modified as justified, to be compatible with existing design bases methods for CR3. The purpose of this criteria is to provide acceptable pipe rupture postulation and protection methods for the plant that in general meet the intent of current NRC requirements, while maintaining and where appropriate, upgrading the existing plant design bases.

The NRC approved pipe rupture report discussed above concluded that the high energy portion of the letdown line outside containment is not subject to a high energy line break. The Standard Review Plan (SRP) allows the establishment of a "No Break Zone" if certain criteria are met. Also, per Generic Letter 87-11 ("Relaxation In Arbitrary Intermediate Pipe Rupture Requirements"), arbitrary intermediate piping breaks need not be postulated if certain criteria are met. The pipe rupture report demonstrates that the high energy portion of the letdown line outside containment meets these criteria. Using SRP methods, the report determined that the piping between the containment penetration and the outboard isolation valve (MUV-49) meets the criteria for a "No Break Zone." Additionally, the report determined that the stress in the piping from the containment to valves MUV-44, MUV-45, and MUV-74 (manual isolation valves downstream of the block orifice and letdown control valves) is low enough that an arbitrary intermediate break need not be considered in this section of the letdown line. Therefore, a break in the high energy portion of the letdown line outside containment is not considered a credible event. As a result, designing for the dynamic or environmental effects of a high energy line break in the letdown line outside containment is not required.

5.4.5 METHODS OF ANALYSIS

5.4.5.1 SEISMIC ANALYSIS OF STRUCTURES

The containment vessel was analyzed by Kalnin's shell program (Ref 29) as described in Section 5.2.4.1.2. The other structures which are not shells of revolution were analyzed by response spectrum method. It was based on a cantilever beam model with mass points chosen at points of mass concentration and connected by mass-free springs. The mass includes that of floor, contributing walls, and heavy equipment. The flexibility matrix or stiffness matrix of the system was generated by STRUDL program and the eigenvalues and eigenvectors were calculated by the method of Jacobi diagonalization. After the participation factors were calculated from mode shapes, the dynamic displacements and accelerations were then computed from the response spectrum value, modes shapes, frequencies, and participation factors. The equivalent inertia forces were applied to the structures statically to determine the internal shears, moments, and support reactions.

The vertical component of ground motion is assumed to be 2/3 of the horizontal one. The criterion used was to combine the responses due to vertical and horizontal input by the absolute sum. Since Florida is in the low seismicity zone, the structural response due to horizontal input is very small compared with allowable and does not control the design. Hence, the structural response due to vertical input was assumed to be insignificant. Stresses and deflection resulting from the combined influence of the normal loads and the additional loads from the 0.05g earthquake were calculated and checked against the limits imposed by the design standard or code. The combined influence due to 0.1g earthquake was checked to verify that deflections do not prevent functioning and that stresses do not produce rupture of excessive distortion.

By engineering judgment, the shear center is close to the mass center of each floor of the structure. Hence, the torsional modes of vibration were considered to be insignificant.

The fixed base mathematical model is shown in Figure 5-28. To prove the conservatism of the fixed base assumption, the results are compared with those of the flexible base model. In deriving the flexible base model, the dynamic in situ soil property is used in calculating spring constants (Ref 41). The soil damping value consists of two parts: material and rotational. The theoretical swaying rotational damping value is very high. However, for conservatism, the sum of the material and the rotational damping value used in the analysis is assumed as small as 5%. The results of the acceleration comparison are as shown in Table 5-6. The comparison indicates that the fixed base assumption is a reasonable and conservative one.

The comparisons of floor response spectra generated by response spectrum method and time history methods are shown in Figure 5-29 and Figure 5-30. The results indicate that response spectrum method is a conservative one. In the time history method, either 1940 N-S component of El Centro or 1952 N21E component of Taft or other recorded time history was used as input. The magnitude of the time history was scaled in such a way that the calculated response spectrum values are at least equal to the ground response spectrum values at the natural frequencies of the structures.

The most important factor which shifts the peak width of the floor response spectrum is the strain dependent soil shear modulus. Since Crystal River Unit 3 is on a rock site, this kind of effect is almost negligible. However, for conservatism, the peak width of the floor response spectrum includes a 10% shift on each side.

The Class I systems and components usually vibrate at the frequency of the dominant mode of the building. When we assume this frequency to be 10 cps and the duration of the earthquake to be 30 seconds, the number of loading cycles is estimated as 300 cycles. This is considered to be trivial. The damping values are listed in Section 5.2.4.1.2. Young's Modulus used for steel is 30×10^6 psi, for concrete for containment vessel it is 4×10^6 psi, and for concrete auxiliary building it is 3.15×10^6 psi. Since the smoothed response spectrum was used as input, variation of material properties causes only slight variation of response.

The natural frequencies and response loads obtained from the seismic system analyses are summarized as follows:

a. Reactor building shell (8 lumped masses)

Natural Frequencies (Hz)
4.4
16.0
30.6
42.3
50.8
60.0
83.3
98.4

Mode Shape and Modal Responses of the Fundamental Mode

Mode Shape	Modal Acceleration (g)	Modal Displacement (ft)
0.054	0.0077	0.323E-3
0.136	0.0194	0.812E-3
0.222	0.0318	0.133E-2
0.310	0.0443	0.185E-2
0.472	0.0674	0.282E-2
0.665	0.0949	0.397E-2
0.851	0.1216	0.508E-2
1.000	0.1428	0.597E-2

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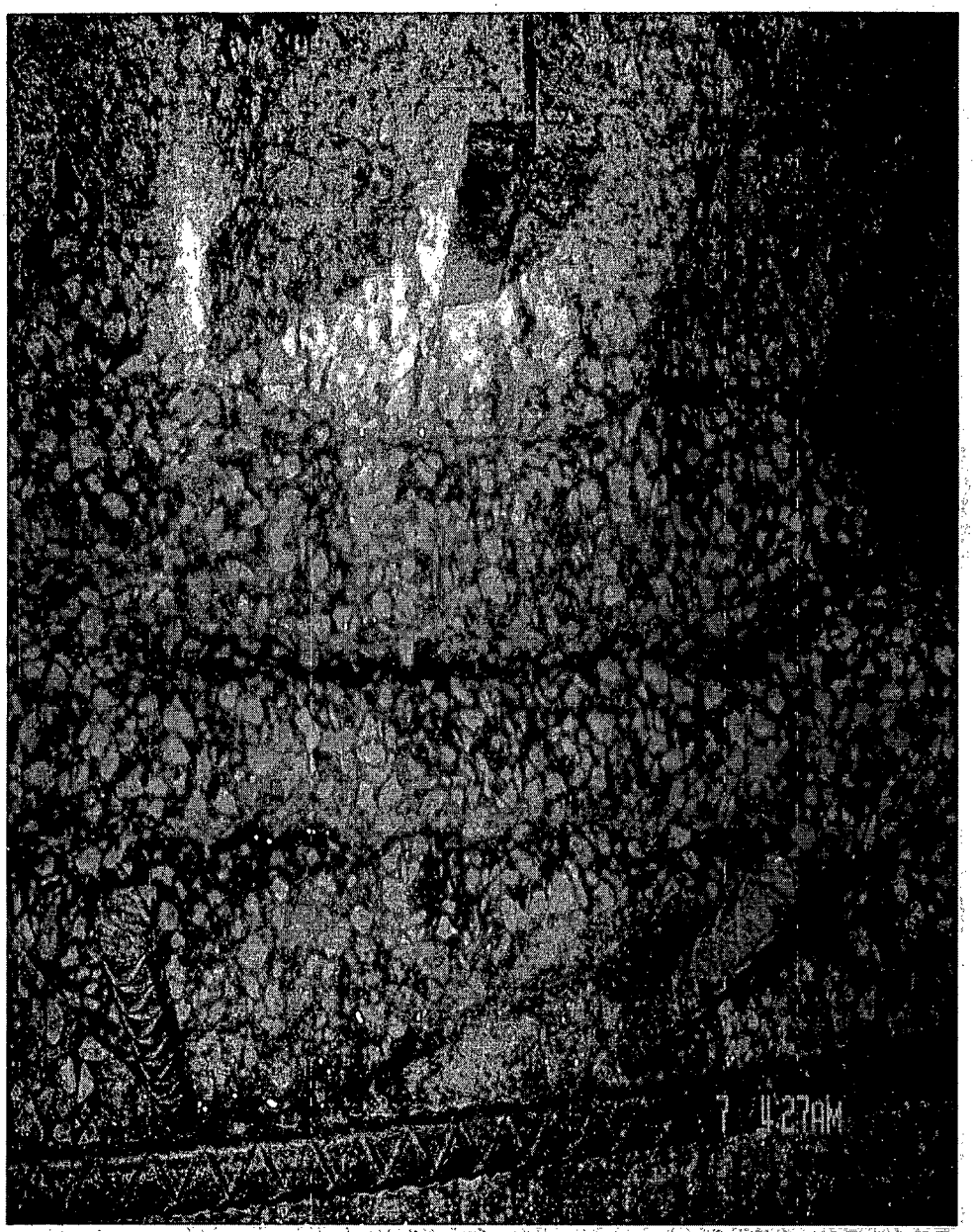
Crack Direction and Path

Multiple cracks could be consistent with vibration-induced fracture



10/30/2009 Interview Rich Kopicki (Progress Energy contractor)

Present: Rich Kopicki, Craig Miller, Patrick Berbon

Rich Kopicki and Richard Ionelli were inspecting the liner because paint flakes were reported. They were inside the RB while hydrolazing was taking place outside.

1- Liner plate

The bulges on the inside of the liner have been attributed to pulling a vacuum inside the containment. Have been there a long time. Present from floor to top of vessel at various locations.

On 10/5, paint scaling was initially reported as being due to scaffold rubbing against it, as a large scaffold was erected in this area. The main flaking point was 1.5ft above scaffold board.

2- Vibration and heat

The next morning (10/6), the paint flaking was clearer and more extensive.

The hydrolazing was reaching the last 3in of concrete and getting down to the liner.

They were on the scaffold and observed the following:

- Liner plate was vibrating (radially);
- Liner plate was warm to the touch;
- Could see the paint shaking and flaking off;

The paint scaling was now observed to follow the location of the vertical stiffeners (present on the other side of the plate). Several areas were vibrating the most, mostly vertically, with at least one instance of a horizontal line.

3- Follow up

Rich took pictures and will send them to Craig Miller.

We will get photographs of the cut portion of the liner plate to mark precisely the positions of high vibration points.

NCR 00360026 - CR3 SGR LINER PLATE COATING SPALLED DURING HYDRO DEMOLITION

A WALKDOWN OF THE LINER PLATE WAS PERFORMED AT 08:30AM ON 10/06/09 WHILE HYDRO-DEMOLITION OF THE CONCRETE WAS IN PROCESS TO REVIEW A CONCERN OF THE NUCLEAR COATING ON THE LINER PLATE SPALLING OFF IN CERTAIN AREAS ON THE LINER. THE VERTICAL SPALL LOCATIONS APPEAR TO COINCIDE WITH THE LOCATION OF THE WELDS ATTACHING THE LINER STIFFENER L 4 X 3 ANGLES TO THE LINER PLATE. THE MINOR HORIZONTAL SPALL APPEARS TO BE AT THE WELD JOINT JOINING THE PLATE SECTIONS. THE LINER PLATE APPEARS TO BE SUBJECTED TO TWO DISTINCT PHENOMENON DURING THE HYDRO DEMOLITION PROCESS. THE FIRST IS THE VIBRATION INDUCED BY THE HIGH PRESSURE WATER JETS HITTING THE CONCRETE AND THE LINER PLATE (THE HIGH FREQUENCY VIBRATIONS WERE CLEARLY FELT BY PLACING A HAND ON THE PLATE). THIS VIBRATION IS RESTRAINED AT WELDED JOINT. THE SECOND IS THE INCREASE IN LOCALIZED THERMAL GROWTH OF THE PLATE DUE TO THE TEMPERATURE RISE DUE TO THE RAPID HYDRAULIC PRESSURE OF COMPRESSION AND EXPANSION IN THE LOCAL AREAS OF ACTIVE WORK. THE PLATE WAS VERY WARM TO THE TOUCH IN THE LOCALIZED WORK AREA BUT COOL TO THE TOUCH IN THE NON WORK AREAS. I BELIEVE THAT THE SPALLING IS DUE TO THE COMBINATION OF BOTH CONDITIONS OCCURRING SIMULTANEOUSLY IN A LOCAL REGION OF THE LINER. THE FLAKING OF THE LINER COATING WILL NOT AFFECT THE INTEGRITY OF THE LINER PLATE.

From: Portmann, Rick
To: Jopling, Daniel L.
Cc: Kopicki, Richard J
Subject: RE: Mysterious Lines on RB Containment Liner
Date: Thursday, October 08, 2009 3:39:44 PM
Attachments: Post Hydro Liner 10-8-09-C.jpg
Post Hydro Liner 10-8-09-A.jpg
Post Hydro Liner 10-8-09-B.jpg
image001.png

New photos from this afternoon. Everyone here says its new and did not exist prior to the hydro-dem. Rick

From: Jopling, Daniel L.
Sent: Thursday, October 08, 2009 2:06 PM
To: Portmann, Rick
Cc: Kopicki, Richard J
Subject: RE: Mysterious Lines on RB Containment Liner

I think it is in that area. Rich Kopicki can better answer the question as he observed the condition first hand.

Dan Jopling
Supervisor
Steam Generator Replacement Project
352 563 2943 X 1759
V net 240 1759
Cell (b)(6)

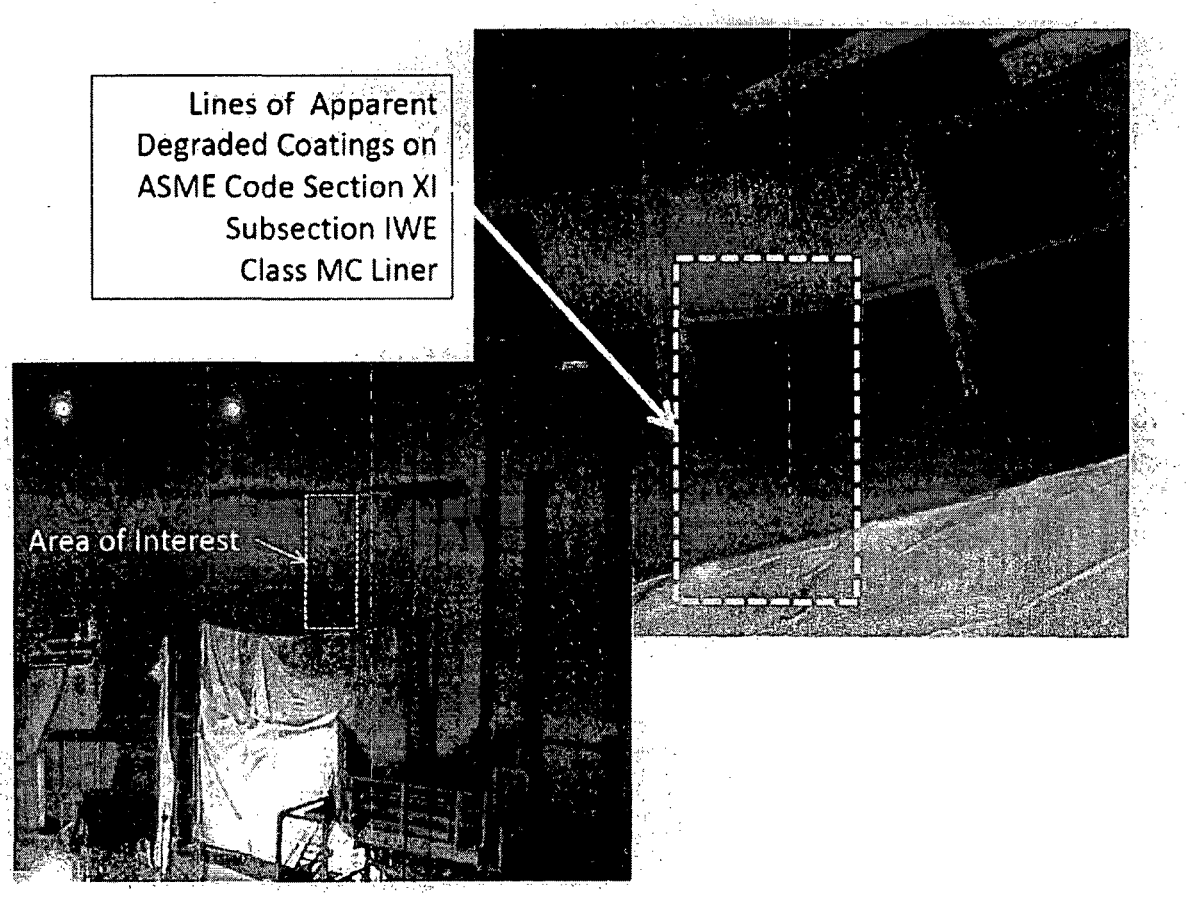
From: Portmann, Rick
Sent: Thursday, October 08, 2009 1:41 PM
To: Holliday, John; Jopling, Daniel L.
Subject: FW: Mysterious Lines on RB Containment Liner
Importance: High

John / Dan - Was this the area you guys looked at earlier? I am having the IWE Inspector who looked at this area prior to the start of the Hydro-Dem. and have him evaluate any differences. Rick

From: Williams, Steven K
Sent: Thursday, October 08, 2009 1:07 PM
To: Mueller, John; Portmann, Rick; Seijas, Donald L.
Cc: Howard, Timothy R
Subject: Mysterious Lines on RB Containment Liner
Importance: High

Based on discussion this morning with Don Seijas, unusual lines of what appears to be degraded coatings on the liner (two lines going up the side of the RB containment) was observed last night. As part of the EGR-NGGC-0023 condition assessment (>180' and Dome areas), I could clearly see these unusual areas with binoculars and was able to take these photos from the top of the elevator shaft (l) and the top of the Rx Vessel Head stairway (r). The shadowing/lighting is very tricky from these vantage points, but it looks like some of the liner may also have some deformation. I did not observe this in R15 (but wasn't looking for it either). I'm sure I would have noticed the degraded coatings along the welds; however, EGR-NGGC-0023 doesn't usually look at the liner since the Containment Inspection Program (IWE/IWL) usually inspects these areas.

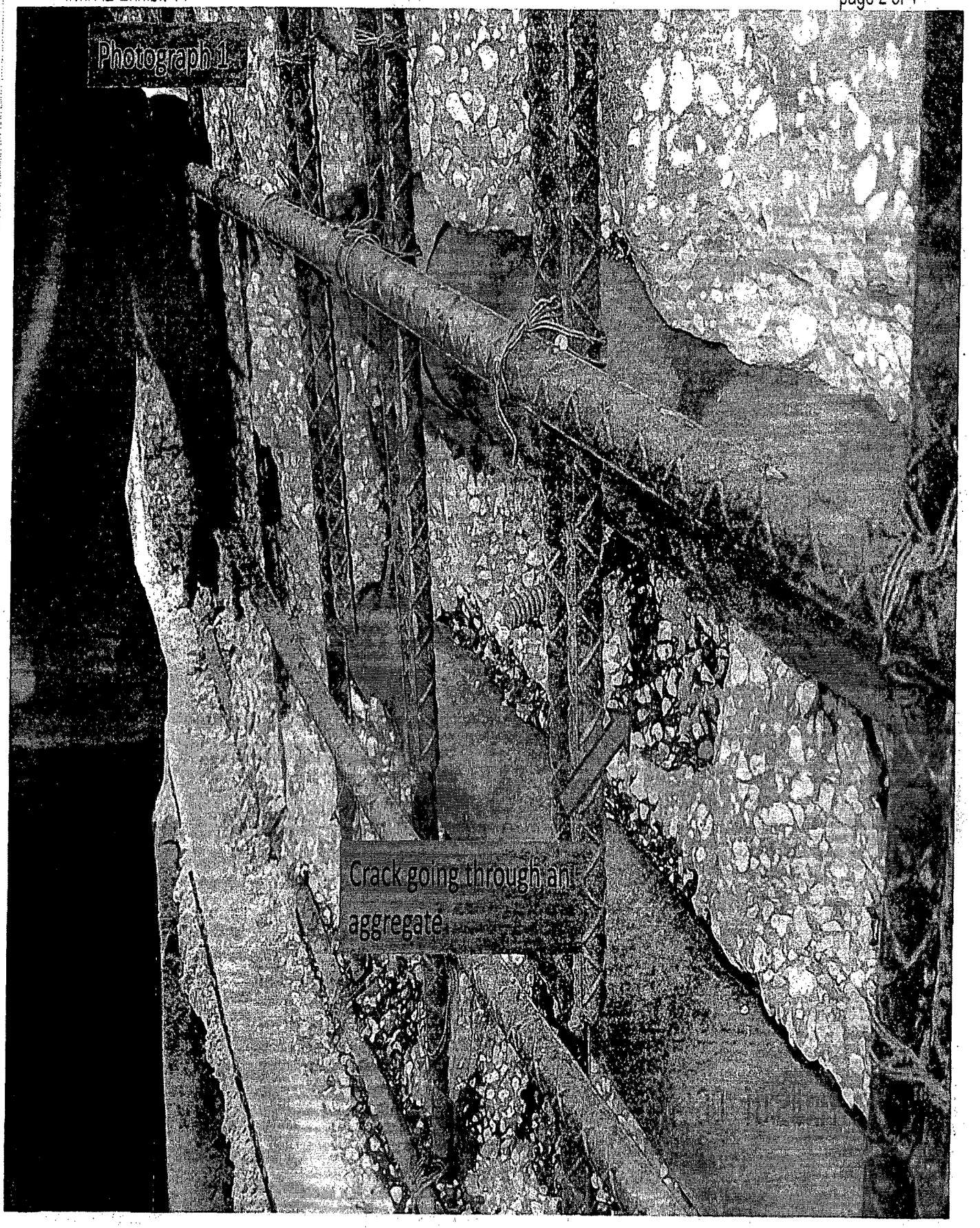
I will need a closer look to make a determination if this needs to be considered an area of unqualified Service Level I coatings. What are the qualification/requirements for me to go up on the top of this scaffolding (Fall Protection, etc.)?



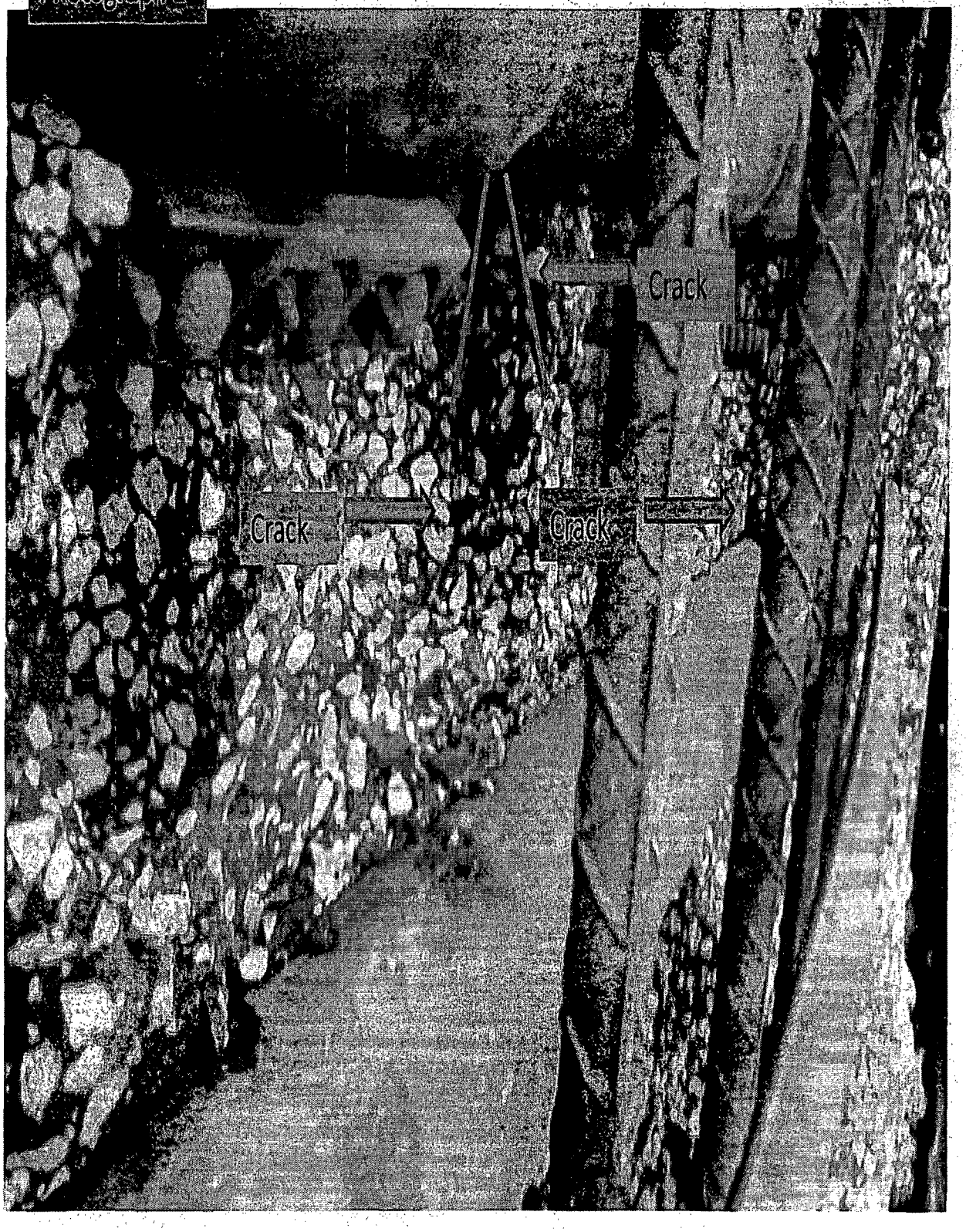
Steven K. Williams
Senior Materials Engineer
BESS – Engineering Material Programs
Progress Energy - Brunswick Nuclear Plant
P.O. Box 10429, BNP 02, Southport, NC 28461
Telephone > 910.457.2318
Pager > 910.412.0845

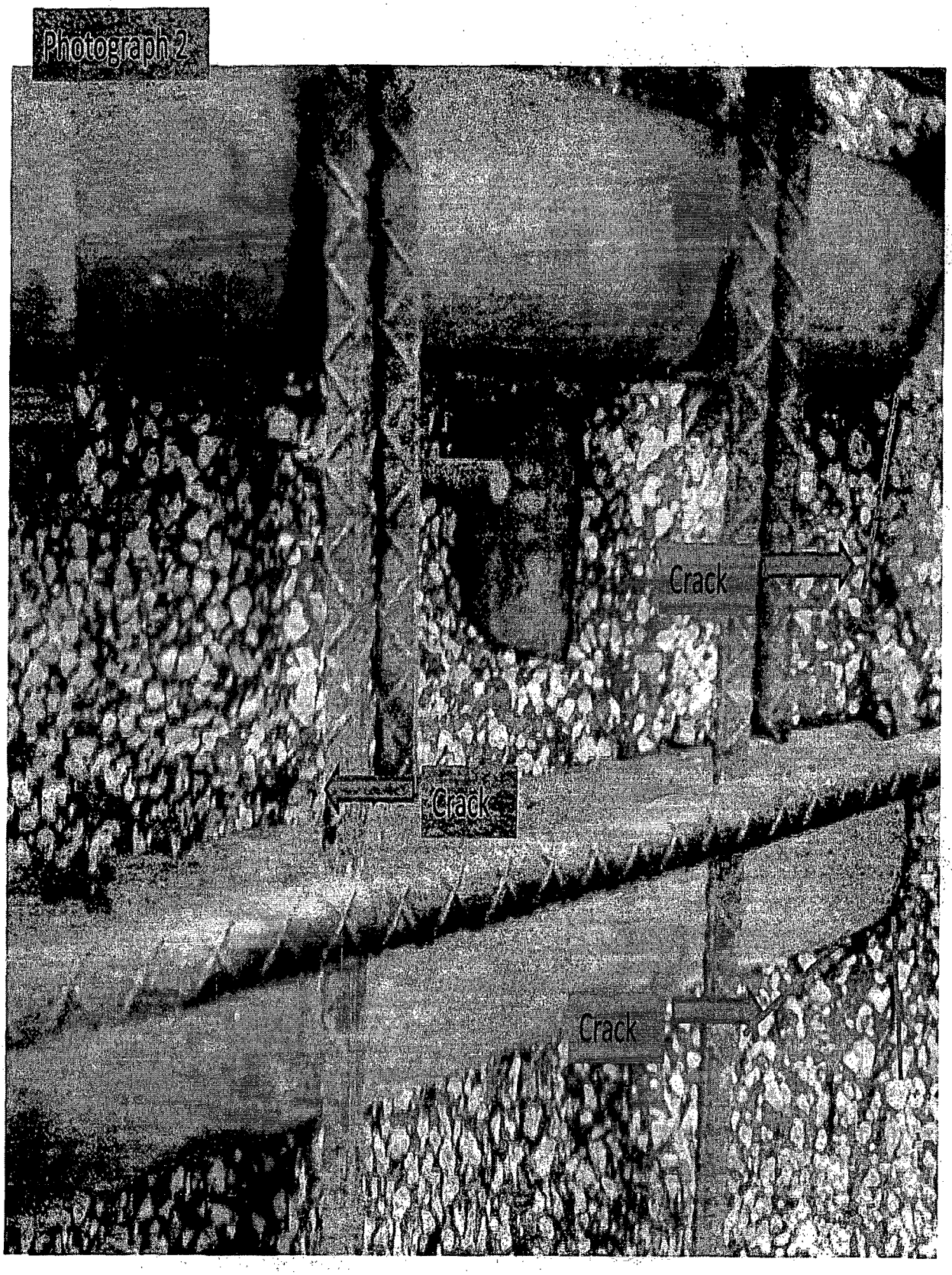
Observation of photographs taken on 09/30.

- Cracks seen on 09/30 at 10:24am on photograph 1;
- Photograph 2 taken on 09/30 at 10:25am (from camera details). Also shows several cracks running from one hoop tendon to the adjacent one;
- Appears to be a V shape from the tendon sleeve;
- If a crack grows from the lower tendon up and another crack grows from the upper tendon down, and they do not grow in the same plane, they will not meet until reaching the adjacent tendon, and we have this V pattern;
- Photograph 3 (taken 9/30 at 12:25am from camera details) shows same cracks;


Photograph 1
Crack going through an aggregate

Photograph 2




Photograph 2
Crack
Crack
Crack

A crack is observed close to the liner plate on 10/07, the day after the end of hydro-blasting:

- Photograph 1: a crack is seen close to the liner plate on the right side of the SGR opening;

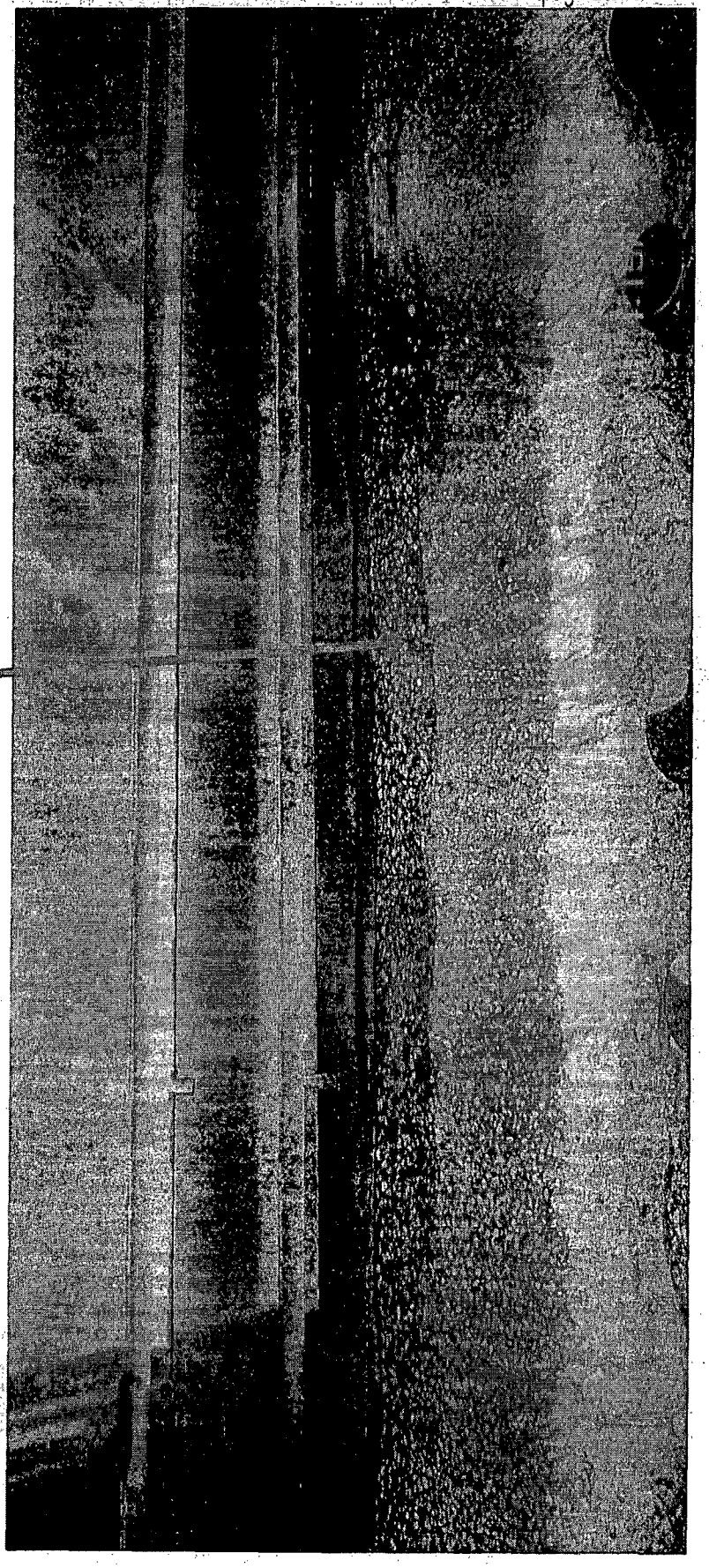
- Photograph 2: the same crack is seen close to the liner plate on the right side of the SGR opening later in the day;

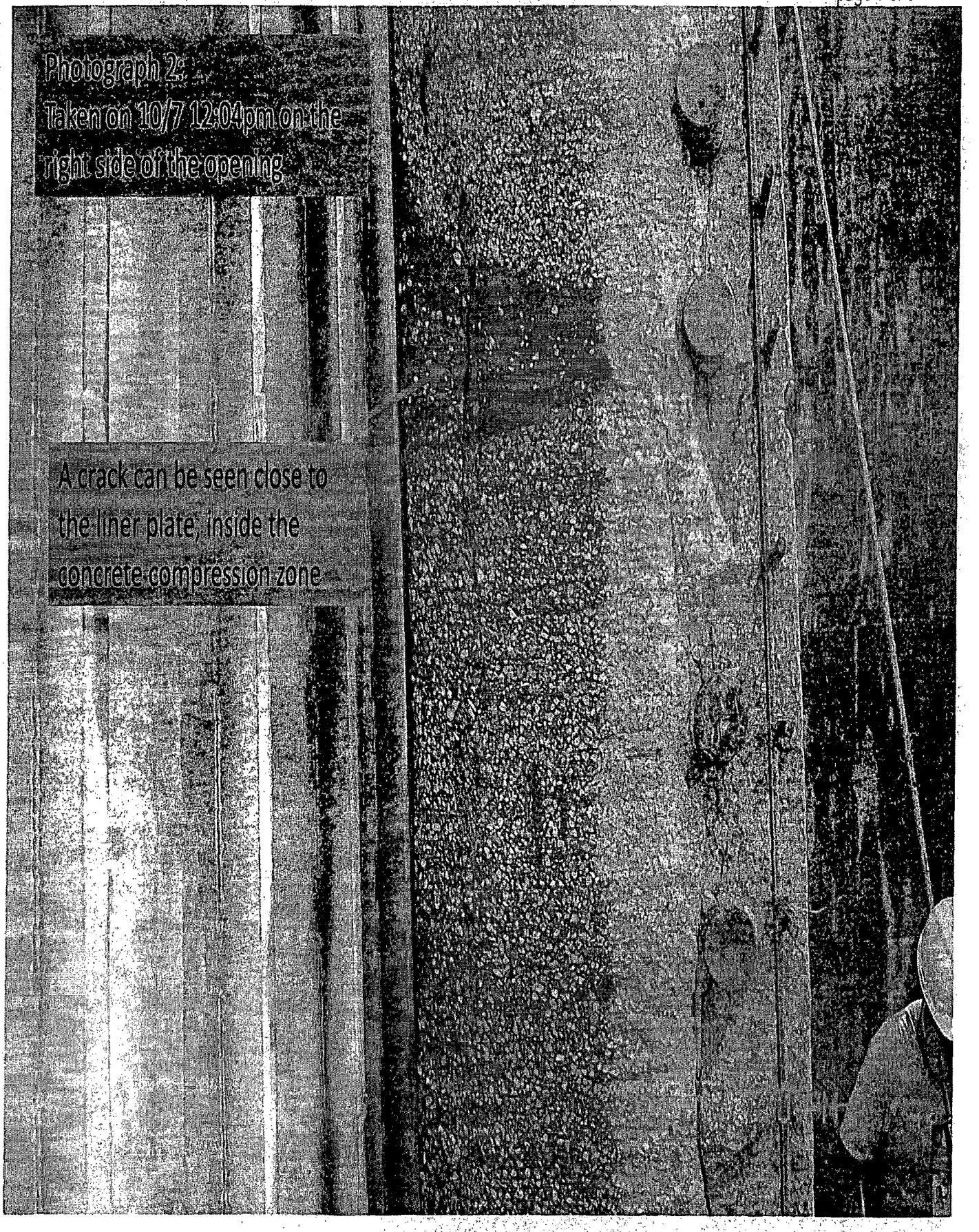
- Photograph 3: the cracked concrete has now fallen down. A liner stiffener is seen right underneath the cracked concrete;

- Photograph 4 : the cracked concrete has now fallen down. A liner stiffener is seen right underneath the cracked concrete;

Photograph 1
Taken on 10/7 4:15am on the right side of the opening

A crack is seen close to the liner plate




Photograph 2:
Taken on 10/7 12:04pm on the right side of the opening
A crack can be seen close to the liner plate, inside the concrete compression zone

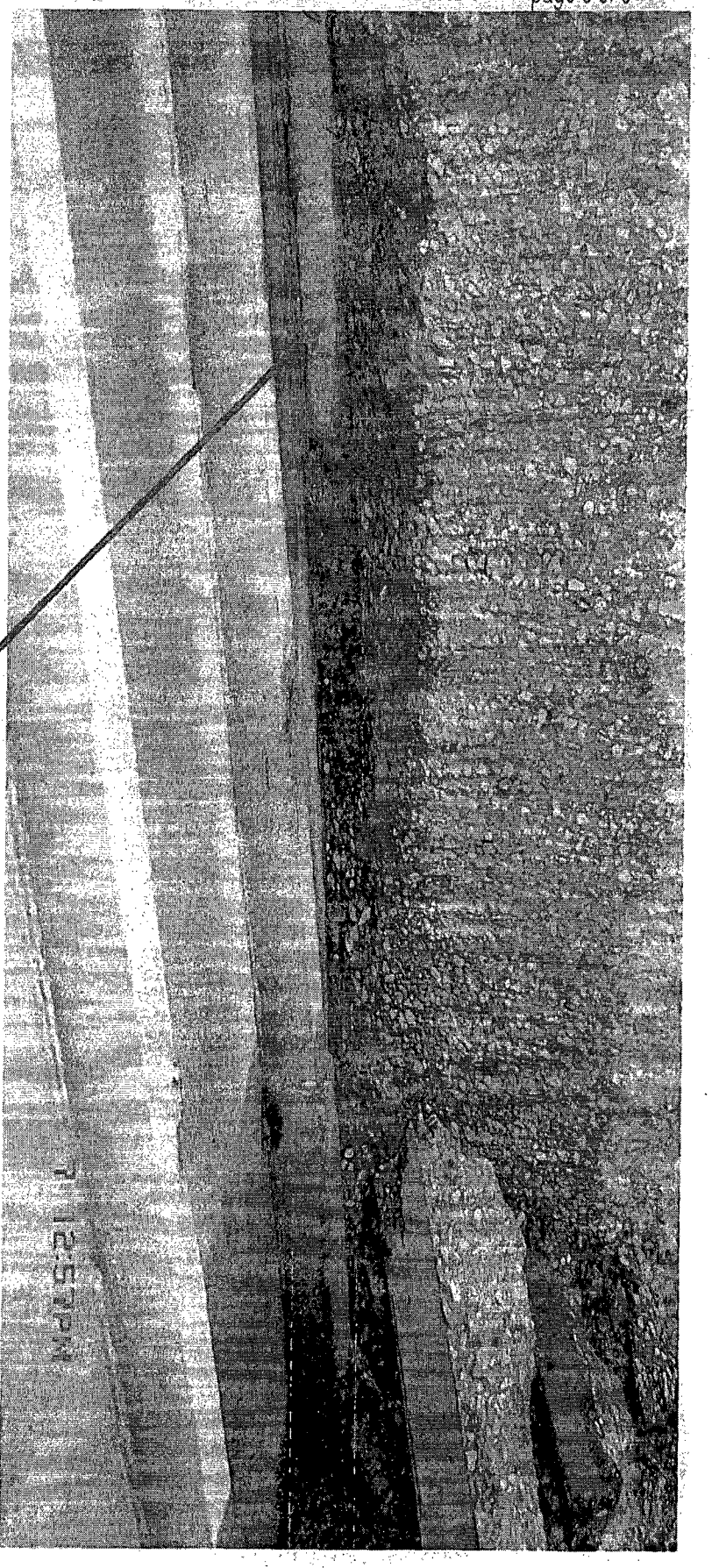
Photograph 3:
Taken on 10/7 12:56pm on the right side of the opening

Cracked concrete is falling down (possibly chipped for safety reasons)

A liner stiffener is found right underneath the cracked concrete

Photograph 4:
Taken on 10/7 12:57pm on the right side of the opening

The piece of cracked concrete has fallen.
A liner stiffener can be seen right underneath the cracked concrete piece



Discussion:

- The impact of the hydro-blasting water jet on the liner and remaining (thin) layer of concrete inside the SGR opening created radial movement;

- The radial movement was transferred to the liner stiffener;

- The stiffener being stronger than the concrete in tension, stiffener movement led to cracking in the concrete;

- The cracking runs perpendicular to the SGR opening surface and curves around the liner stiffener to contact the liner right behind the stiffener;

- We clearly observe a stiffener right underneath the cracked concrete;