7. *Complete/partial destruction/removal of roofing/deck:*
If large sections of the roof and deck have been removed by the storm, the key to a determination of causation is the flood elevation. If the roof is low and evidence exists of floatable debris in the roof joists, consider flood as having created a buoyant force, facilitated by floating debris, to pop the roof from the joists. If the flood elevation is significantly below the roof and no floatable debris is seen anywhere on the roof joists, then consider wind as the cause. A word of caution is offered here, though. In one instance, so-called "floatable debris" was seen lodged above the roof joists in one corner of a structure whose roof had been completely removed in the storm, but evidence elsewhere on the site could only document a flood elevation of 2 ½ to 3 feet. Close examination of the "floatable debris" by high-resolution digital photography disclosed that the material in question was a pre-incident bird's nest, originally built between the roof deck and the ceiling and accessed via an opening under the eave.

8. *Buckling or translation of vertical and horizontal girt system:*
Girt systems may be seen commonly as either set atop a masonry wall, or as extending from the roof to the ground. Such girts are usually sized only to reinforce and support light-gauge metal siding, and are not considered as "stand alone" portions of the structure. Deformations to such systems in the form of a local buckling of cold-formed members or of a translation of an entire wall section have been noted in both Plaquemines and St. Bernard Parishes. Again, the height of the flood can be used to index a likely causation. If situated atop a masonry wall, the cause is nearly conclusively wind. If extending from roof to ground and associated with other flood-induced damage, the conclusion suggested is as strongly in favor of water as the cause.

9. *Isolated penetration damage to exterior walls:*
Isolated penetration damage to exterior walls used to be clearly attributable, as the object inflicting the damage could be seen still lying there. With debris removal operations far advanced in some areas, the object(s) may have been carted away. As a suggestion, consider the size and location of the penetration. If large and located at or below the documented flood elevation, it was likely caused by floating debris, and the amount of energy needed to cause the penetration is a clue as to the rough speed of the advancing water. If small, or if located above the flood elevation, consider wind-borne debris as the causative agent.

10. *Failure of interior partitions:*
Unless there is a direct route to the failed partition from outside the building afforded by other wall and partition damage, failed interior partitions can nearly always be plausibly attributed to flood. The mechanism may be a softening of the wall materials and subsequent gravity-load failure, a buildup

of hydrostatic water to one side and not the other (as discussed above), hydrodynamic (surge) forces, water-borne floating debris, or a collateral manifestation of hydrostatic uplift on the slab below the partition. To date, there have been no structures inspected by this writer where a failed interior partition, as an isolated manifestation, was attributable to wind.

11. *Translation of unanchored exterior accessory equipment:*
In one or two interesting cases, there have been significant translations of outside accessory equipment (notably a trash compactor) as a result of the storm. In the case of the compactor, evidence of the "skidding" of the steel support legs in the dirt told a story of flooding, buoyancy of the compactor, wind on the high compactor hopper catching the wind like a sail, breaching of the compactor throat to flood the ram and box, and re-settlement of the compactor at some distance from its original location. Other examples of this can be anticipated with other accessory equipment whose buoyancy is either later abridged (as in this case) or whose buoyancy is sufficient to just clear the ground with later re-settlement caused by abating flood water elevations.

12. *Failure of all or most of exterior masonry infill:*
If evidence of significant wind-induced deformation of the building frame is found and if the masonry infill is not competently anchored to this frame, consider that wind-induced racking may have "cracked out" the rigid infill as a result of distortion of the infilled portal. If there are other evidences of flood-induced damage and if the masonry is well-anchored to the building frame, consider a hydrostatic load of flood water as a defensible cause.

13. *Pattern breakage of exterior glass windows:*
Normally, pattern breakage of glass in a well-ventilated building is considered to have been caused by hydrostatic or extensive water-borne debris forces. Causation in the absence of evidence of hydrostatic load (i.e. low flood elevation) was discussed above. Any discontinuities of pattern should be checked to determine if unbroken glass is similar in thickness and age to the broken panes. Check, also, to verify that they are actually glass. In one instance, unbroken "glass," after inspection, was discovered to be plastic.

14. *Pattern dispersal of debris including interior contents:*
If debris dispersion is seen to be strongly directional, consider flood as a major factor in the overall damage to the structure. The speed and direction of water tends to be less variable than wind, thus wind-borne debris is more widely scattered and les directional, whereas debris transported by flowing water, unless acted upon by wind forces as discussed above, tends to be strongly dispersed downstream of the flow. Keep in mind that the direction of this flow, in small areas, may be impacted by the presence of barriers to

the water, so deviations from a strict "one-direction" translation are not only possible, but are likely.

15. *Helical distortion of site vegetation, especially large trees:*
In at least one instance so far, evidence of an axial twisting of major trees on a site all in the same direction was noted in the form of a helical twisting of the fibers of the trunk. In addition, other trees of similar size and identical species on the adjacent property were unaffected. This has been judged as evidence of tornadic activity, a condition which has been more completely discussed above.

16. *Uplift and rippling of interior wood flooring, especially gymnasia and civic centers:*
In several institutions, hardwood flooring has been observed to have been lifted and/or buckled in a direction at right angles to the direction of the grain. This is conclusively caused by water, as the buoyant properties of the wood made it susceptible to hydrostatic uplift and the absorbent qualities of the lignin between the grain fibers predispose it to a much greater swelling in the cross-grain direction than the longitudinal.

17. *Translation of exterior steel or aluminum fenestration:*
In all cases so far inspected, translation of doors and windows has been determined to be the effect of flood. The direction of translation is an indication of flooding direction and the timing of the flooding of the interior. Keep in mind that equal elevations of water on both sides of a barrier will, theoretically, produce no net force in either direction. There are undoubtedly exceptions to this, such as the failure of a full-story window system as a result of wind racking of the structure, but this has not been observed to date.

18. *Base rotation and/or mast distortions of tall and slender structures such as light poles:*
If there is no rotation of the base, the height of the mast deformation must be the clue as to cause. If above the flood elevation, then wind is the likely cause. If below, then flood OR wind may be at fault, but likely flood, as wind forces increase with height, and a higher bend location would be expected. If a base rotation is present, mast bend must be carefully documented, as the base rotation may give the appearance of bend where none exists. Base rotations attributable to flood can occur if saturated alluvial soils permit a rotation of a deep footing or shallow piles. For this reason, examine the base for signs of cracking and the base of the mast for signs of impact from water-borne debris. If the base is cracked, and no impact point is seen, wind is the likely agent. If the mast is uncracked and/or an impact point can be identified, consider flood as having softened the soils and/or conveyed heavy debris to the mast to cause the rotation.

19. *Shredding of thin-segment overhead roller doors:*
Several sites have been inspected where thin-segment overhead roller doors have been shredded by stripping segments from one another and fatiguing the material at a batten or splice point. This has been attributed, so far, exclusively to wind, as the fatigued metal would have to undergo cyclic flagellation as discussed above. Examine the ends of such segments for evidence of cyclical bending in both directions.

20. *Translation of entire portions of wooden structures remaining structurally intact:*
Sometimes, entire roof structures have been seen, intact, several feet from the original site. If the nature of the roofing material is such that wind removal would be expected, examine the underside of the roof perimeter to determine the means by which it was affixed to the top of the wall. If the connection appears to have been competent, consider flood as having failed the walls and translated the roof structure by floatation to its present location. If the connection appears weak, consider that wind acting on the gable end may have acted to dislodge the roof without exerting sufficient force to remove roofing. If the roof were easily removed by the wind, there may well have been insufficient resistance to the wind by the connection to cause the shingles to be stripped from this direction.

References:

Grateful acknowledgement is made for the contributions of the following to the assembly of a picture of the Katrina time-dependent dynamics:

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SOUTH CLEAR WELL ROOF COLLAPSE:
Hydraulic Uplift or Excessive Construction Loading ?

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Abstract: This paper presents a failure investigation case history of a partial roof collapse of a below-grade water storage facility during rehabilitation. The evaluation considers the original design and maintenance of the facility, rehabilitation design, and construction sequencing. The cause of the roof collapse was the failure of selected columns under approved construction equipment loading. The columns that failed were initially damaged by hydrostatic uplift of the base slab during a severe rainfall event and subsequently loaded repetitively by construction operations. Hydraulic and structural models developed during the evaluation accurately predicted the response of hydraulic systems, general crack patterns and specific crack locations; thereby confirming the failure mechanism.

Introduction

This paper presents the results of our investigation of the collapse of a portion of the roof of the South Clear Well at the Lake Huron Water Treatment Plant in Fort Gratiot Township, Michigan. The South Clear Well is a 330–foot long by 370–foot wide, approximately 17-foot deep, reinforced concrete structure designed to retain approximately 15 million gallons of filtered water. The clear well contains an 18-foot wide influent channel along its east wall and a beam and post support system for future transmission piping in the northernmost 45 feet. The balance of the clear well construction consists of a 12-inch thick floor slab, 16-inch diameter columns with top capitals and bottom pedestals at a 22-foot center-to-center spacing, and a 10-inch thick roof slab. It is located south of the high lift pump station and west of the filter building.

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Improvements to the clear wells and the high lift suction well were designed in 1995. The construction of the improvements began in 1997. The collapse of a portion of the South Clear Well roof occurred at approximately 2:00 p.m. (EDT) on June 22, 1999 during final topsoil placement operations on the roof of the clear well. No persons were injured or construction equipment damaged by the failure.

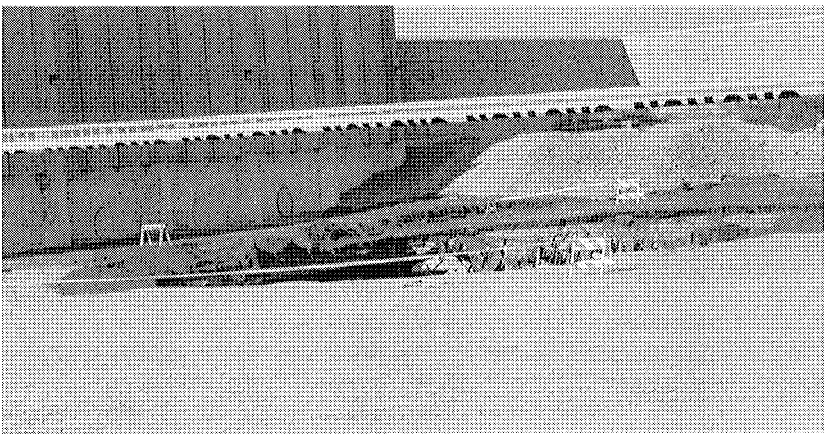

Figure 1. View of collapsed roof section

The evening of the collapse, the plant suffered a power outage and dewatering pumps within the gate wells in the clear well were rendered inoperable. As a result, approximately 10 inches of water from gate leakage covered the floor of the clear well during the initial post-collapse inspections by Detroit Water and Sewerage Department (DWSD), contractor, and consultant personnel. The initial inspection teams stayed primarily under the beam and post section of the clear well out of concern for safety. The two primary observations during the initial inspections were the water appeared to be shallower in the center of the clear well and the most severe roof and column damage had generally occurred beneath the areas of top soil placement on the roof of the clear well.

DWSD requested that NTH Consultants, Ltd. lead a team of local consultants to perform an independent evaluation to determine the cause of the collapse and develop schedule and budget for replacement options. At the request of DWSD, the firms of Nehil-Sivak and Greeley and Hansen were retained to provide structural engineering services and an operational assessment of the plant, respectively. Greeley and Hansen also performed an evaluation of the storm water and under drain systems.

Investigation Methodology

In order to develop cause and effect relationships between the various factors involved in the collapse, a chronological approach was used to create an understanding of the project. We reviewed and evaluated available information on the construction including: the original construction plans; current construction basis of design reports, plans, and specifications; historical maintenance and repair records for the clear wells; available subsurface data; daily field reports; interviews of resident engineering and plant personnel; and measurements and observations made during visits to the site after the failure.

We developed simplified assessments of short term and long term loading conditions for comparison with the original design and evaluated the effect of construction procedures and adjacent construction activities on the basin. We then reconstructed the sequence of events during the construction period and ascertained potential causes of the failure. Based on various load combinations, we then evaluated the structural adequacy of as-designed and as-built conditions to resist the forces associated with the existing conditions at the time of failure.

Original Clear Well Construction

The south clear well was constructed in the 1970's in an open cut excavation with the high lift building and the identical north clear well. The base and roof slabs have thickened sections at the perimeter walls that transition to the typical thickness over a distance of 4 to 5 feet from the face of the exterior walls. The beam and post supported section of the clear well with thickened roof and base slabs also transitions to the typical thickness in the general clear well area over a distance of 4 feet from the face of the beam and post support.

The top of base slab elevation for the clear wells and high lift suction well are 587.5 and 579.5, respectively. The original under drain system, installed to prevent groundwater accumulation beneath the base slab, combines relief wells adjacent to north and south sides of the high lift building under the clear well base slabs, a 3-inch thick sand drainage blanket under most of the clear well, collector piping under the clear well at invert elevation 584.5, and perimeter collector pipes backfilled with pea gravel around the clear wells at invert elevations ranging from El. 584.5 for the north clear well to 587.5 at the southeast corner of the south clear well. All under drain piping was originally connected to a dedicated under drain pump station located at the northwest corner of the north clear well. A review of the original construction drawings indicates the high lift suction well, the clear well areas beneath the beam and post system, and the exterior clear well walls do not have a sand drainage blanket and are not serviced by the under drain system.

Planned Rehabilitation

The purpose of Project No. 4, Water Storage Reservoir Improvements, was to eliminate the problem of surface water ponding on top of the reservoirs. The ponded water potentially could leak through the soil cover and into the reservoirs. The basis of design report recommends the regrading of the surface of the clear wells with lightweight concrete fill to promote drainage; installation of a polyethylene liner over the top of the lightweight fill; construction of storm sewers and french drains to collect surface runoff and infiltration; construction of new north and south storm water detention basins; construction of new north and south pump stations to handle combined surface and under drain flow; abandonment of the existing under drain pump station; repair of cracks, construction joints, and control joints in the interior roof slabs, walls and base slabs; gate repair; installation of new air vents; and repair of clear well roof hatches.

The calculations for the new storm water collection system, detention basin, and pump station indicate the north and south systems are sized for a 10-year storm at maximum operating conditions. The pumps in each pump station were sized to provide a total 800-gpm capacity and are controlled to maintain a 10-minute pumping cycle time at minimum operating conditions of 250 gpm.

Rehabilitation Plans and Specifications

The contract plans and specifications were consistent with the basis of design report. To address potential hydrostatic uplift during construction, the specifications required phased construction of the north pump station, demolition of the dedicated under drain pump station, dewatering and rehabilitation of the north clear well, construction of the south pump station, and dewatering and rehabilitation of the south clear well. The contract documents required dewatering of the clear wells for a period of 6 months over two consecutive winter seasons to facilitate the repair work in the basins. The additional requirement to remove all surface backfill to place the new drainage system reduces the dead weight available to resist hydrostatic uplift forces.

To address the potential for construction equipment overloading, the plans contain a note prohibiting the contractor from placing more than 225 psf of load on the roof of the reservoir including the proposed lightweight concrete fill, soil, and equipment.

Shop Drawing and Construction Change Documentation

The contractor performed independent structural calculations and a Cat 936 end-loader with a three cubic yard bucket and 42,000 pound truck was approved for soil removal/filling operations. The loaded trucks would only travel on the beam and post supported section of the clear well and empty trucks could be driven on the remainder of the slab only if the soil was removed.

Construction Chronology

At the completion of activities at the north clear well, DWSD took possession of the north clear well and turned the south clear well over to the contractor. At the time of the collapse, the north pumping station was pumping the surface drainage from the north and the under drainage from the north and south clear wells. The south pump station was not operational and the temporary surface drainage was accomplished by submersible pumps placed in the manholes of the new storm water collection system.

The soil removal operation was conducted without incident using the same methods used on the north clear well and interior and exterior crack repairs were completed in late March, 1999. DWSD plant and field engineering personnel conducted an inspection of the reservoir for cleanliness at the completion of the work within the clear well. During the crack sealing work and periodic inspections, while the clear well was well lit, no signs of structural distress were noted in the floors, walls, roof or columns of the south clear well. After March 29, 1999, entry into the clear well was limited to maintenance of dewatering pumps at the northwest and southeast corners.

The lightweight concrete placement on the roof commenced on April 28, 1999. The contractor pumped and shaped the lightweight concrete with no equipment on the roof. Membrane placement across the roof of the clear well was accomplished using a forklift. Sand placement took place working from the southwest corner to the north and east. A protective board was placed over the membrane prior to placing sand with the Cat 936 loader. A D-5 bulldozer with low-pressure tracks was used to grade the sand. Topsoil placement took place working from the middle of the beam and post supported sections at the north end of the clear well. Loaded trucks backed down the ramp from the east and dumped the topsoil on the thickened roof slab section. The Cat 936 loader retrieved the topsoil and spread it over the basin.

On June 22, 1999, topsoil had been placed over approximately the east half of the clear well. The loader operator noted a small hole had developed in the roof and soil was running into the clear well. The operator moved the loader off of the roof of the clear well prior to the collapse at approximately 2:00 p.m. (EDT).

Post Failure Field Observations

<u>Initial Inspection</u> - The team noted that six columns had collapsed under the failed roof section. All six collapsed columns appeared to have broken off at the connection between the column and the capitals/pedestals. The column capitals and pedestals were still connected to the roof and floor slabs, respectively.

Numerous columns outside of the collapsed roof area were also damaged. Concrete spalling was noted at the top and bottom interface between the column and the capital/pedestal on opposite sides of the columns. Some severely damaged columns had been displaced on shear cracks running diagonally through the column at the top. Severely damaged columns were concentrated in areas with similar crack orientation.

No new cracks were noted in the roof or the walls. Previously repaired cracks appeared to be intact. Some displaced floor cracks were noted parallel to the face of the wall, but the depth of water and sediment on the clear well floor made it impossible to map.

<u>Subsequent Detailed Inspections</u> - Inspection teams confirmed the observations made during the initial inspections and also discovered a pattern of damage consisting of:

- The collapsed portion of the roof had separated from the clear well on the east, north, and west sides and was still connected on the south edge north of column line 6f (See Fig. 2). The remaining roof slab on the north edge of the collapsed area lines up with a family of previously grouted roof cracks approximately four feet from the face of the beam and post supported section (See Fig. 3). The remaining roof slab on the east and west edges of the collapsed area extends from the edge of column capital up to four feet from the capitals. The lower mat of roof reinforcing bars were stripped from the underside of the remaining slab in the east, north and west sides (See Fig. 4). It appears the northern edge of the collapsed roof section broke free and encountered the clear well floor first.

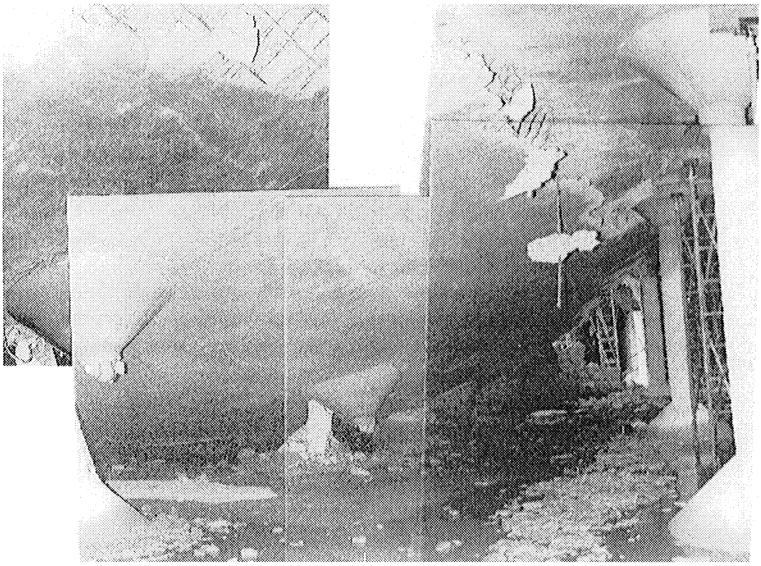

Fig 2. Hinge formed at south edge of collapsed roof section at column line 6f

Fig 3. Collapse along existing cracks

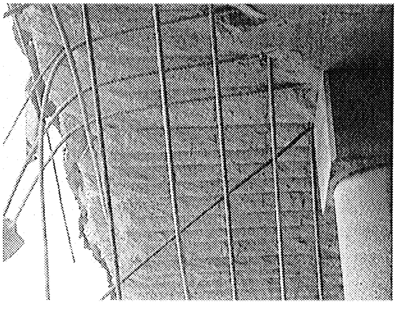

Fig 4. Roof slab steel stripped