PERFORMANCE OF UNDERGROUND STRUCTURES AT THE JOSEPH JENSEN FILTRATION PLANT

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SYNOPSIS

The San Fernando, California, earthquake of February 9, 1971, caused considerable damage to structures, utilities, equipment, etc., and has been the source of numerous publications and technical papers. Damage to the structurally completed underground portions of the Jensen Filtration Plant was extensive and resulted in structural failures among the most spectacular of the San Fernando earthquake from an engineering point of view. The paper describes the most significant of these failures of underground structures, discusses the causes of the damage and indicates general approaches and details which should reduce this type of structural failure in future seismic activity.

INTRODUCTION

The Joseph Jensen Filtration Plant is located northwest of the Van Norman Lakes. The plant's structures occupy an area approximately 4000 feet by 1500 feet near the base of the San Gabriel Mountains. At the time of the earthquake the plant was approaching the completion of construction and consisted of numerous basins, tanks, buildings and underground conduits normally associated with water treatment plants and a large underground finished water reservoir. The plant is owned by the Metropolitan Water District of Southern California and will process water delivered from Northern California by the California State Water System. The site is near the edge of an alluvial plain in the northwest portion of the San Fernando Valley. The soils at the site consist of compacted fill, loose natural soils and dense natural soils of the Sangus formation. Liquefaction of loose, saturated natural soils resulted in extensive landsliding along the eastern edge as well as some localized effects elsewhere on the site. A discussion of the liquefaction effects on the affected structures is beyond the scope of this paper. A brief mention might be in order, however, of the Main Control Building, a three story structural steel framed building with reinforced concrete shear walls and a heavy grid foundation. This building settled 6 to 8 inches, 2 inches differentially, and moved horizontally over 4 inches due to the liquefaction induced landslide. It sustained only a few very fine cracks illustrating the excellent performance of this type of structure and foundation system.

The site has been estimated to be approximately five miles from the zone of maximum energy release of the February 9, 1971, earthquake. The Richter 6.6 earthquake caused estimated maximum accelerations in rock at the site of approximately 0.3g with maximum surface accelerations estimated at approximately 0.4g in the Sangus and 0.5g in the area of deepest compacted fill. Estimated spectral peak accelerations were two to three times the maximum ground surface acceleration in the period range of 0.3 to 0.6 seconds.

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This paper will discuss three specific underground structures, the large Finished Water Reservoir, the double box Effluent Conduit and a Box Culvert supporting a railroad spur, which were among the most heavily damaged structures.

FINISHED WATER RESERVOIR

The Finished Water Reservoir is a large, underground structure of reinforced concrete. The reservoir is 520 feet by 500 feet in size and varies from about 17 feet deep at the perimeter to 35 feet deep in the central portion, see Figure Number 1. The roof and floor slabs are both of flat slab construction with drop panels and column capitals, the roof slab being 14 inches thick while the floor slab is 16 inches thick. The perimeter walls are 14 inches thick at the top and taper to 21 inches thick at the floor slab. Twenty-four inch diameter columns are spaced at 20 feet on center each way. Construction joints are located at 20 feet on center each way midway between columns in the roof and floor slabs as well as the exterior walls. At the time of the earthquake, the concrete work of the reservoir was complete except for the small access structure on the roof. The reservoir was to be covered with 8 feet of fill to counteract the buoyant pressure of the ground water. The water table was lowered by pumping to 3 to 5 feet below the floor slab during construction. Fill was in the process of being placed on the roof and was about 50 percent in place, varying from about 6 feet deep on the north end to very little on the south end, when the earthquake occurred.

Structural damage to the reservoir was very extensive and spectacular. The west wall failed at its base for over 400 feet, see Photograph Number 1. The actual failure took place at the top of the reinforcing steel dowels, about 2 feet 3 inches above the floor slab drop panel, Photograph Number 2. As the wall failed the earth pressure pushed the wall into the reservoir and it dropped, lowering the roof slab over 2 feet. The east exterior wall also failed at its base and moved into the reservoir as much as 2 inches. Furthermore, the east wall had numerous horizontal cracks, was bowed inward as much as 4 or 5 inches, and had a massive wall failure near the north end.

There were numerous failures in the roof slab construction joints, principally at lines 24-1/2, B-1/2, 3-1/2, 7-1/2, and 8-1/2. (Our designation of line 24-1/2, for example, refers to the construction joint located midway between column lines 24 and 25. Line B-1/2 is similarily located between column lines B and C.) The distress on line 24-1/2 was the most dramatic failure where a 5-foot wide zone of concrete had spalled for the full 520-foot length of the construction joint and all reinforcing bars were stretched, see Photograph Number 3. At several locations along line 24-1/2 pockets of sound concrete stretched the reinforcing bars to a point where they failed in tension (see Photograph Number 4) as the slab on each side of the construction joint slid back and forth. Measurements taken indicate that the slab to the south of the construction joint moved with respect to the slab on the north side a minimum of 15 inches to the east and 11 inches to the west. These measurements reflect the final location of the bars and do not account for the snap-back or rebound of the bars after they had broken. Photograph Number 5 is a close-up showing several of these reinforcing bars which had necked down and failed in tension. Note the scratch marks on the concrete where the deformations on the reinforcing bars had actually clawed at the concrete during these large movements back and forth of the roof slab at the construction joint at line 24-1/2. Close examination revealed that the top of deformations on the reinforcing bars were somewhat worn and small flecks of the steel could be observed adhered to the spalled concrete surface. The roof slab joints at lines 3-1/2, 7-1/2, and 8-1/2 probably moved back and forth only an inch or two as compared to the 26 inches of measured movement on line 24-1/2. The roof joint on line B-1/2 also had sizable back and forth movements, but not as large as on line 24-1/2. The walls at the ends of the massive roof failure on line 24-1/2 also experienced spectacular failures. Photograph number 6 shows the failure at the west end as well as the failure at the wall base. Photograph number 7 shows the east wall failure and how the wall was deformed by the sliding of the roof slab along the construction joint. The roof slab near the east wall from line 24-1/2 south to the outlet structure failed approximately 6-1/2 feet from the face of the east wall. The roof slab near the north wall failed in a similar manner for the western half of the reservoir. Photograph number 8 shows a portion of this failure along the north wall, and illustrates how the slab failure generally followed the ends of the negative reinforcing between the column and middle bands. Approximately 80 percent of the reinforced concrete columns of the reservoir are spalled near their two ends, indicating movements compatible with the roof failures. Cantilevered baffle wall columns were out of plumb with one of them collapsing. Major cracking and damage resulted at the outlet structure. Spalled concrete also resulted at numerous construction joints in the floor slab, and the reinforcing bars crossing several of these joints were very slightly bent.

The reservoir was subjected to complex ground motions and forces during the earthquake of February 9, 1971. The exact failure mechanism is difficult to define, but the following failure mechanism and sequence of failure have been reached based on a detailed study of the resulting damage. The reservoir roof slab, the fill on top of the roof slab, the walls, columns, floor slab, and the earth against the reservoir walls were all subjected to earthquake-induced inertia forces, resulting in high in-plane forces in the roof diaphragm as it transferred shears to the perimeter shear walls. The fact that the fill was mostly in place on the north half of the reservoir induced higher inertia forces in that portion of the reservoir. The shearing forces in the reservoir roof clearly exceeded the ultimate strength of the construction joints of the roof slab, which were continuously keyed for vertical loads but did not have intermittent shear keys nor diagonally placed dowels. Based on the damage pattern, it is the authors' opinion that the roof slab failure was initiated by east-west inertia forces at line 24-1/2 which is parallel to and 30 feet from the north wall as shown in photographs 3, 4 and 5. Naturally earthquake forces occurred simultaneously in the north-south direction and produced subsequent failure of the roof construction joint along line B-1/2 which extended the entire 500-foot length. Examination of the damage indicated that the movement along the line B-1/2 joint was considerably less than along line 24-1/2. The resulting displacement of over 2 feet of the roof slab south of line 24-1/2 caused very high passive earth pressures against the exterior east and west walls. These passive earth pressures resulted in the subsequent failures of the east and west walls. Considerable torsional response or rotation of the reservoir roof slab occurred after these initial failures which shifted the center of rigidity from the center to the southeast corner of the reservoir. As the earthquake progressed the torsional components increased the shearing stresses in the southern portion of the reservoir and resulted in the roof failures on line 3-1/2 and then on lines 7-1/2 and 8-1/2. This rotation of the roof slab induced large displacements and, therefore, greater damage in the north end of the east and west walls which is consistent with the damage. Likewise, the forces and movements at the west end of the north and south walls would be greatest, and the damage followed this same pattern. The direction and amount of spalling on the columns are also consistent with this damage sequence and roof slab rotation.

The failure in the roof slab adjacent to the north and east walls, as shown in Photograph Number 8, is also a result of these large horizontal movements of the roof slab which created large bending moments in these slabs near the exterior walls. The combination of the large bending moments together with the weight of the slab and earth fill and the fact that the slab steel reinforcing greatly decreases between the quarter and third point of the slab caused the resulting failures of the roof slabs at these locations. The heavy damage and shattering to the buttress-type piers of the outlet structure probably occurred late in the seismic response of the structure after the roof failure initiated on line 3-1/2. At that time, the outlet structure was the most rigid element connected to the central area of the roof diaphragm. The roof slab failures along lines 7-1/2 and 8-1/2 undoubtedly occurred shortly thereafter.

The numerous construction joints in the floor slab which spalled appear to be the result of several causes. The inertia forces of the roof were eventually transferred to floor slab which caused high in-plane forces within the slab. Furthermore, a reported eastward movement of a few inches of the reservoir plus some localized liquefaction underneath the reservoir undoubtedly caused high forces in the floor slab. These forces and movements would certainly affect the floor slab and appear to be a major factor causing this distress. It is the authors' opinion that the localized liquefaction and the reported small permanent movements of the reservoir caused very limited structural damage and were not a significant factor in the overall damage to the structure.

EFFLUENT CONDUIT

The Effluent Conduit is a buried reinforced concrete double box culvert which runs in a north-south direction for approximately 2000 feet. Each barrel of the double box structure is 8 feet 6 inches wide with the height varying from 7 to 8 feet. The alignment of the conduit is straight except for a gentle "S" curve approximately opposite the north end of the reservoir. The conduit was completed and backfilled at the time of the earthquake with approximately 20 feet of fill for most of its length reducing to 10 feet opposite the reservoir. The conduit walls are 12 and 15 inches thick with #5 bars at 12 inch centers each way each face except the outer vertical bars which are #6 bars at 6 inch centers. The roof and floor slabs are 15 and 16 inches thick and are reinforced with #7, #8 or #9 bars at 12 inch centers at various cross sections. The base of the conduit rests on natural soils of varying density. After the earthquake the backfill was removed for over 500 feet to expose the heaviest damage. The damage varies from minor at the north end to moderate at the start of the "S" curve, severe in the "S" curve area, moderate south of the "S" curve, and severe at the junction of the conduit and the reservoir inlet structure. The conduit generally experienced differential displacements and/or rotations at most expansion joints. Horizontal cracking occurred near the top and bottom of all three walls. This cracking was minor in the northern portions of the conduit, but was severe in the "S" curve area where it increased to spalling and fracturing of the walls as the roof slab moved to the east, see Photograph Number 9. There were longitudinal horizontal cracks near the middle of the exterior walls and at the top of the foundation dowels at some locations. Approximately 500 feet of the box structure had been "racked" 4 inches maximum out of plumb to the east, see Photograph Number 10.

In general, the lateral racking that developed was the result of high inertia forces and displacements developed in the soils adjacent to and above the conduit. Seismic shaking produces a tendency for differential lateral displacements over the height of a buried structure. If these displacements are large enough, then racking and failure of the buried structure can occur, depending on the characteristics and ductility of the structure. The magnitude of the displacement depends on the soil conditions with greater relative displacements tending to occur in looser soils. Racking of the conduit resulted in severe cracking and spalling of the walls. Other factors, such as the soil profile under the conduit, some localized liquefaction, and the alignment also affected the structural performance of the Effluent Conduit, but it is the authors' opinion that the primary cause of damage was the differential lateral displacements of the surrounding soils

RAILROAD SPUR BOX CULVERT

The box culvert for the railroad spur track is a skewed four-barrel, reinforced concrete box culvert, 118 feet in length. Each barrel is 12 feet wide, 8 feet 6 inches deep, and has a north-south oriented axis. The top and bottom slabs are 16 and 18 inches thick, respectively, while the interior walls are 8 inches thick with #5 bars vertically each face at 8 inch centers and the exterior walls are 10 inches thick with #7 bars vertically each face at 12 inch centers. The embankment fill on top of the structure was about 16 feet deep, two-thirds of the design depth, at the time of the earthquake. The culvert was half full of mud and silt at the time of the earthquake, see Photograph Number 11. The five 118 foot long walls are all damaged at their tops and in many cases also at the bases. Photograph Number 12 is a view of the top of one of the interior walls. All walls of the culvert failed due to lateral racking in an east-west direction. The roof slab moved 4 to 6 inches east with respect to the base slab. In some cases, the walls failed and offset at the top while in other cases the wall failed both top and bottom and leaned full height to the east.

The box culvert is a partially buried structure with the west abutment having more extensive backfill then the east abutment. Earthquake induced inertia forces of the structure and embankment fill caused lateral movement toward the unconfined eastern edge. The inertia forces combined with differential lateral earth pressures and displacements resulted in the lateral racking and failure of all the walls.

RECOMMENDATIONS FOR IMPROVED PERFORMANCE

Many conclusions and recommendations can be drawn from the failures of the underground structures at the Jensen Plant. Perhaps the most elementary conclusion is that underground structures must be designed for seismic forces. Current building codes in the United States do not require such design for buried structures. It is the authors' opinion that this design must be based on two parameters - force and displacement. The structure must be strong enough to resist the inertia forces generated within the structure. In the case of the reservoir, the mass of the earth fill on the roof increased the inertia forces on the reservoir and such earth masses must be included as a design factor. Consideration of the displacement of the structure as well as the displacement of the surrounding soil is equally important to strength. The structural engineer must keep in mind that the inertia forces commonly used in structural design are generated by ground displacements. In soil masses the seismic displacements vary with depth, being greatest at the ground surface. Loose soils tend to exhibit greater displacements than similar depths of firmer soils. These differential soil displacements cause differential lateral displacements of a buried structure. The structure must have sufficient flexibility and ductility to undergo these displacements without severe damage.

Equally important to design forces and displacements are construction details. A factor which has a significant effect on the seismic performance of any concrete structure is the detail of the construction joints. The construction joint details of the reservoir were a continuous key which will increase the shear resistance for loads perpendicular to the slab or wall but not for seismic shears parallel to the joint within the plane of the slab or wall. A construction joint is a weakened plane through the structure. Concrete is a material which shrinks and creeps with age. This shrinkage and creep causes cracking within the concrete, and, at construction joints where there is a discontinuity, a fine crack or gap is generally created. When seismic forces are applied it is necessary to transfer the in-plane seismic shears across the construction joints which probably do not have the concrete surfaces in contact. The only resistance is then the continuous reinforcing bars acting as dowels which is inadequate for the high forces generated. This situation has caused many engineers designing structures in seismic areas to provide intermittent keys in <u>all</u> construction joints to act as lugs and increase the resistance to in-plane shear forces at construction joints. Some engineers have similarily used diagonal reinforcing steel dowels. However, the reservoir did not have any intermittent shear keys in the construction joints and this omission was a significant contribution to its spectacular failure. The joint on line 24-1/2, 520 feet long, slid over 26 inches back and forth! If adequate construction joint details had been provided, it is the authors' opinion that the massive failures of the reservoir that occurred would have been prevented or substantially reduced, although some cracking and minor damage would still have resulted.

ACKNOWLEDGEMENT

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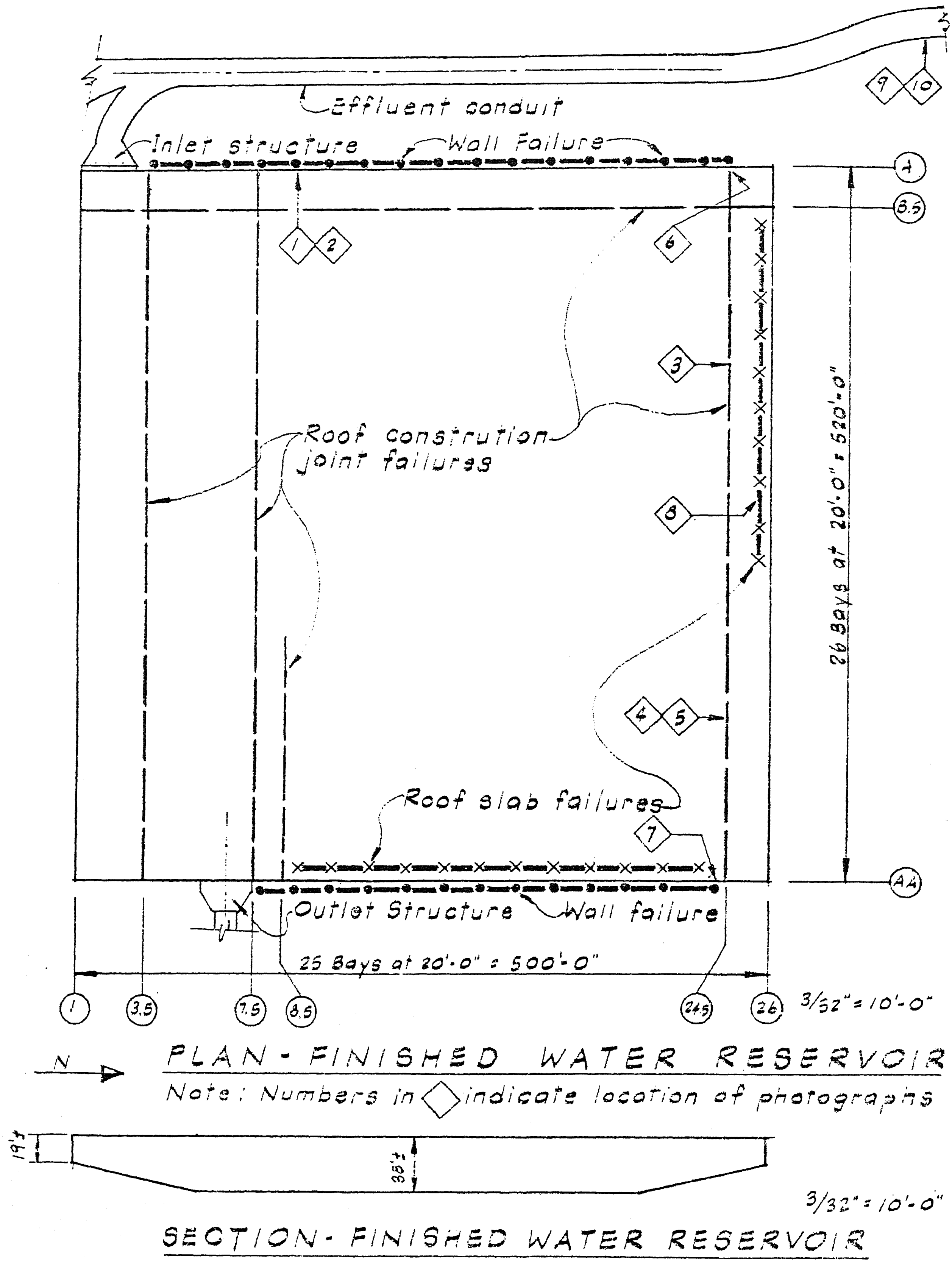
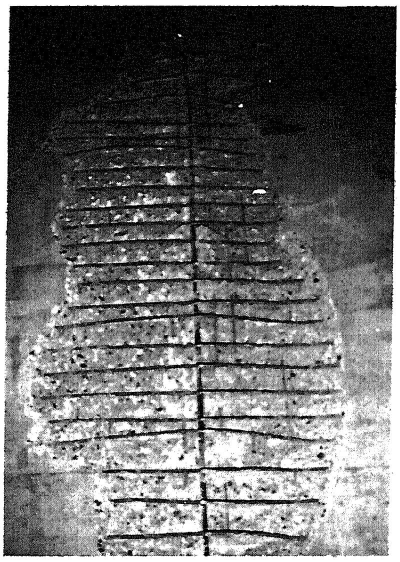


Figure Number 1
Finished Water Reservoir, plan and section

Photograph Number 1. West wall of reservoir failed at its base. Wall has pushed in and dropped. Concrete in foreground is floor slab drop panel.

Photograph Number 2. Close-up of west wall failure at base of wall. Rebar is inner face wall reinforcing. Horizontal break in concrete occurs at top of dowels.



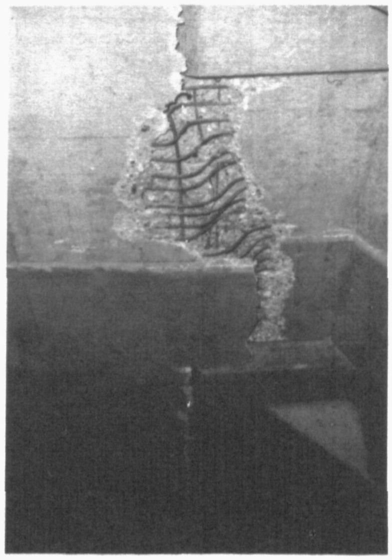
Photograph Number 3. Typical view of construction joint on line 24-1/2. Width of spalled concrete is approximately 5 feet wide. Note how reinforcing bars have been stretched.

Photograph Number 4. Construction joint on line 24-1/2 where three reinforcing bars failed by necking in tension. Measurements indicated that the joint slid a minimum of 15 inches one way, 11 inches the other way.

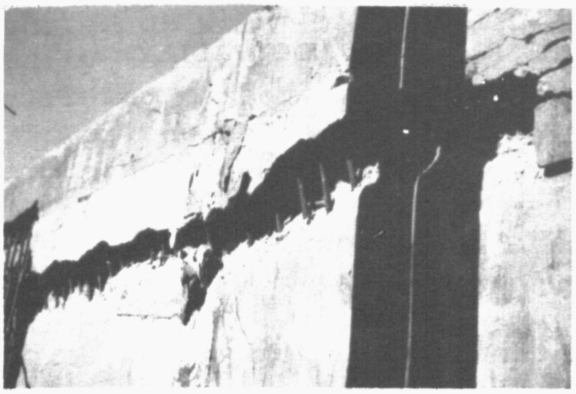
Photograph Number 5. Close-up of broken reinforcing bars in Photo Number 4. Note tension failures of reinforcing and scratch marks of deformations on bars on the concrete surface.

Photograph Number 6. West wall of reservoir at line 24-1/2, looking south. Wall has moved inward 3 feet and dropped about 2 feet.

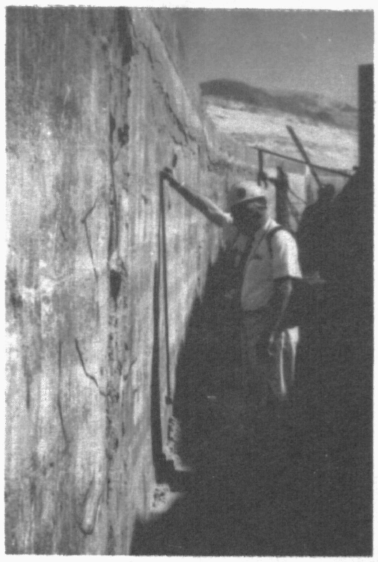
Photograph Number 7. East wall of reservoir near line 24 looking north. Wall at left is cantilever overflow wier wall. Note slab failure above.



Photograph Number 8. Roof slab failure adjacent to north wall. Note how failure occurred at end of top reinforcing bars which were shorter in middle band between drop panels.

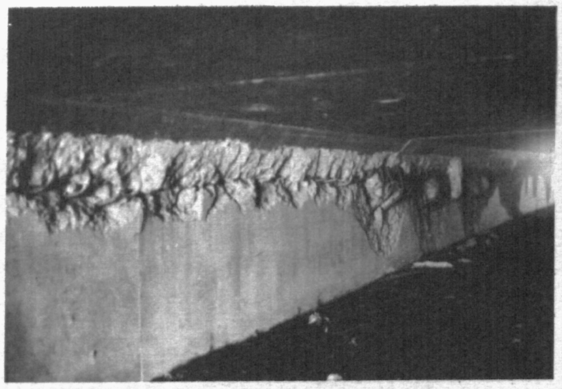


Photograph Number 9. East wall of double box Effluent Conduit after backfill removed. This was the point of heaviest damage.



Photograph Number 10. East of wall of Effluent Conduit in "S" curve area after back-fill removed. Note out of plumb of wall, which was 4 inches maximum for 8-1/2 foot inside height.

Photograph Number 11. Box Culvert for railroad spur line looking north. Culvert is half full of mud and silt. Culvert racked to east or right by 4 to 6 inches.



Photograph Number 12. Interior wall of Box Culvert which failed at its top.