"WATERTIGHT RELIABILITY OF
CONDENSATE STORAGE TANK AND
ITS CONCRETE ENCLOSURE WALLS
UNDER OBE AND TORNADO EVENTS"

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Abstract

The methodology and results of calculations are presented to demonstrate that the Condensate Storage Tank and its surrounding enclosure walls constitute a reliable storage facility to satisfy emergency cooling water requirements. The structural integrity, watertightness and water recovery aspects of the storage facility are verified with respect to the Category I Seismic event (DBE) and the Tornado event postulated for SONGS 2 & 3.

1.0 Introduction

The SONGS 2 & 3 facility incorporates a condensate storage capacity of 650,000 gallons per Unit, provided by two steel tanks (T-121 and T-120, Figure 1.1). The Primary Condensate Storage Tank has a capacity of 150,000 gallons and is a seismic category I, tornado resistant facility. The tank is fully capable of providing all the water required for a normal shutdown sequence. The second tank is a 50 foot diameter by 39 foot high conventional steel tank with a rated capacity of 500,000 gallons. This tank is enclosed in a concrete structure with a 4 foot thick base mat and 2 foot thick plane walls extending to a 30-foot height above the base, as illustrated in Figure 1.2 and 1.3.

According to the criteria dictated for the postulated emergency coolant requirement, 200,000 gallons of fresh water must be assured over a 24-hour period in addition to the 150,000 gallons currently provided by the Primary Condensate Storage Tank. These additional 200,000 gallons are expected to be provided by the existing Condensate Storage Tank and its enclosure structure subject to upgrading structural modifications to be defined herein.

A summary of the analytical results and design modifications that form the basis for the effectiveness of the water storage facility is presented next, and a more detailed description of the analytical procedures is covered in sections 2.0, 3.0, and 4.0.

1.1 Seismic Event

1. The existing seismic class II tank is not adequate to withstand a seismic class I event. The salient findings are that the base anchorage of the tank is deficient, and that the lateral seismic

response of the tank is high and subject to uncertain analytical determination because of the buckling mode of failure predicated for the thin vertical tank walls under high compressive stresses due to overturning moment. Therefore, (1) the base anchorage will be augumented by a new auxiliary concrete mat that interlocks the tank base with the surrounding walls and improves the uplift resistance at the tank base, and (2) a seismic restraint for the tank will be provided at the top of the walls in order to restrict the upper lateral response of the tank and preclude adverse interaction with the adjacent enclosure walls.[a] The "tight" restraint will eliminate the development of impact loads, and will deliver the lateral loads to the walls as "in-plane" loads for efficient utilization of the ample shear wall load capacity.

2. The foregoing measures, while definitely effective to restrict the seismic response of the tank and prevent damage to the surrounding concrete structure, may not assure the watertight integrity of the tank. Therefore, the extreme case of total spillage of the tank water contents into the concrete enclosure will also be considered as a possible occurrence under a seismic event. In this case the concrete enclosure will be relied upon to retain the required water.

3. Justification of the water-retaining ability of the concrete enclosure is based on (1) verification of its structural integrity under the prescribed seismic event, and (2) evaluation of leakage rate resulting from concrete cracking due to maximum response and residual loading after the seismic event.

   A structural analysis of walls by finite element modelling was performed considering hydrostatic loading, and the seismic inertia and hydrodynamic loadings derived from dynamic analyses.

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a. The restraint would not be provided only if no adverse tank/wall interaction were to be demonstrated upon further non-linear rigorous analyses of the tank seismic response.

To obtain a bounded solution, the analysis was performed considering both the minimum and maximum water levels in the tank. The maximum is considered to be the full 500,000 gallon capacity of the tank while the minimum contents is 280,000 gallons, which will be enforced by plant technical operation specifications. The results indicate that (1) the structural integrity of the walls as originally designed is adequate in both cases and (2) there are limited zones of high flexural stresses with attendant concrete cracking.

The leakage calculation is based on the crack pattern evaluated under sustained hydrostatic loading taking into consideration the effects of the initial loading and the deformation undergone by the walls during maximum seismic response; see Figures 4.2, 4.3 and 4.4 for crack patterns. The corresponding calculated leakage proves to be insignificant and totally acceptable for both water levels as shown in Table 1.1. The crack widths in zones below the top water level and above the base of the wall where a continuous waterstop is provided are smaller than the maximum 0.004 in. crack width recommended by the ACI[1] for water-retaining structures. The ACI criterion is referenced only for correlation and comparison of the order of magnitude of the calculated crack widths; it is not used as the basis for concluding that the structure is watertight. The most severe cracking results at the joint of the base mat with the wall where, in the case of maximum storage, the residual crack widths are substantially larger than the ACI recommended maximum width. However, the condition is not adverse because (1) a flexible waterstop and shear key are provided at that joint as well as at all other joints, and (2) even if the leakage associated with the larger cracks is calculated considering the extreme case of a totally ineffective waterstop, the resultant leakage calculated would be acceptable, as summarized in Table 1.1.

1.2 Tornado Event

1. The existing thin shelled tank is vulnerable to tornado missile perforation at the locations not shielded by the concrete

enclosure walls. However, the adequacy of the tank is maintained based on consideration of (1) the governing missile trajectory and the angle of incidence dictated by the geometry of the tank with respect to the enclosure walls, and (2) the aggregate thickness of steel plate and travel through water that the missile must undergo before impacting the rearmost plate. These considerations demonstrate that the lower level of the tank corresponding to 220,000 gals. is not susceptible to perforation, see Figure 1.3. Therefore the minimum 200,000 gals. required plus the inside-tank unrecoverable allowance of 20,000 gals. are retained as debris-free water inside the tank, and there is no need to rely on water in the annulus between the tank and its enclosure.

2. The limited exposure of the steel tank above the enclosure walls renders the tank adequate for maximum positive wind pressure due to tornado event. The loading due to transient depressurization under tornado event will be reduced to an acceptable limit by incorporating venting modifications into the tank, thus rendering it adequate for all tornado pressure loadings.

Table 1.1
Leakage Evaluation (Gallons)

Initial Water Contents of Tank (A)	Minimum Required plus Unrecoverable Water (B)	Allowable Water Leakage (A) - (B)	Maximum Estimated Leakage Under Several Postulated Cases, over the Prescribed 24-Hour Period	
			Leakage	Case
500,000 (Maximum specified capacity of tank)	240,000	260,000	1,500	(1) Normal, credible case. Based on effective cracking counting on the reliability of water-stop and recognizing the high quality class I concrete construction which is physically evident from the completed structures.
			40,000	(2) Same as (1), but considering the extreme of total failure of the waterstop at the locations of worst cracking at the base of all walls.
280,000 (Minimum water contents enforced by technical operation specification)	240,000	40,000	150	(4) Extreme case for low water level considering aggravated cracking due to seismic response and failed waterstop.

2.0 Structural Evaluation and Analytical Model

The structural analysis of the enclosure walls was performed with a finite element analytical model of the relevant portions of the structure as indicated in Figure 2.1. Wall (a) was emphasized analytically since its long horizontal span renders it as governing in terms of flexural response and crack formation. Wall (b) was also specifically analyzed and considered to be a conservative representation of the remaining walls which were not subject to specific analysis.

The boundary conditions were selected to obtain the most accurate evaluation of the displacement response undergone by the walls since it was recognized that flexural deflection and curvature are of vital importance in the calculation of crack widths. Accordingly, the rotational boundary condition at the vertical corners of walls (a) and (b) are appropriately represented by extending the model to include the connecting cross walls and introducing the fixed boundary at the far end of these walls. (One edge of wall (b) is assumed fixed, with this fixity substantiated by the two cross walls (d) and (e) and the short span of wall (d)). The base boundary condition of all walls was also considered fixed, which is substantiated by the 4 ft. thick base mat and its overburden load from the heavy tank contents.

Consistent with the necessity to obtain an accurate and conservative evaluation of the wall deformations, a relatively fine mesh of finite elements with reduced effective moments of inertia was adopted for the model. The element moments of inertia were reduced to recognize the flexural section that results upon concrete cracking, and were calculated in accordance with ACI Code 318-71, section 9.3.2.2. According to the Code formulation, the effective moment of inertia is a function of the actual flexural moment developed which in turn depends on the moment of inertia used. Therefore an iterative procedure was implemented and the ultimate result were deflections and curvatures which are conservatively higher than those initially obtained using the moment of inertia of the gross concrete section.

The foregoing considerations (boundary conditions, fine mesh and reduced moment of inertia), plus the proven reliability of plate finite elements used to analyze simple, flat wall structural systems, permitted a high confidence level in the stress analysis and evaluation of wall deformation. The dominant structural behavior is two-way flexural action of the flat walls under pressure loading, with practically no tensile membrane behavior except for the slight horizontal tension developed at the walls' upper levels above the waterline. It follows that the load-induced concrete cracks of concern are dictated by the flexural behavior and corresponding curvature, both of which are highly predictable and accurately evaluated analytically.

The loadings considered were hydrostatic, and seismic loads due to inertial response of walls and hydrodynamic effects due to convective and impulsive fluid pressures derived per reference (2). Three-component earthquake responses were considered, even though under the governing outward pressure loadings, there is no additive effect for the two horizontal components since the maximum out-of-plane displacement response of a single wall, at a given time, is the governing response. The hydrodynamic pressures from the earthquake components were combined by the SRSS. The resultant hydrodynamic pressure was combined with the seismic response of the walls by absolute summation as a conservative recognition of the long-period, "sustained" type of response characteristic of seismic sloshing of liquids.

3.0 Determination of Crack Widths

The crack widths were evaluated using two different approaches, and the higher of the calculated values derived from the more rigorous approach was used in the leakage calculation.

In the first approach, the crack width is formulated as a function of the tensile stress in the reinforcing steel using an equation derived from experimental correlations. The formulation used is per ACI Committee 224[1], which in turn is an adaptation of the original research by Nawy[3] & [4].

The total crack width as obtained from the dominant flexural and the lesser axial tension stresses was calculated. The flexural stresses were determined by linear elastic analysis of the reinforced concrete section using working stress design (WSD) formulations. This is a conservative evaluation for sections subject to moments below the ultimate capacity, whereas for sections at or near the ultimate yielding moment, the appropriate ultimate strength design (USD) formulations were used. Axial stresses were found to be generally insignificant since the in-plane membrane tensions developed in this type of flat-wall flexural systems are normally low. The only locations with some axial tension were toward the top of the walls where horizontal tension related to a slight "hoop" action results, and the stresses were simply calculated as the tension load divided over the total area of horizontal reinforcing (< 1 ksi).

It is recognized that the stress dependent formulations for crack width are approximate and appropriate mainly for an evaluation of cracks by comparison of calculated values against maximum crack widths recommended in ACI 224 for various types of structures. For water retaining structures, a maximum crack width of 0.004 in. is recommended. This limit is basically satisfied by the crack widths calculated by the stress dependent formulation under the sustained hydrostatic loading throughout the walls. The only instances where the maximum width is exceeded are inconsequential because they are at locations above the water level, and at the base of the walls where a continuous waterstop is provided. Therefore, the stress-dependent evaluation of cracks indicates a watertight condition, but it is recognized that the calculation is approximate and that a more direct verification of watertightness addressing the leakage derived from a more rigorous evaluation of cracks is necessary to establish a definite conclusion. The results of this analysis are provided in Section 4.0.

The second approach for the calculation of crack widths is based on the curvature or rotational deformation undergone by the flexural elements. The basic postulation is that the rotational deformation, which is highly predictable and well defined by the flexural action undergone by the walls, is totally achieved through a concentrated rotation assigned to be effected at the postulated crack. This is a conservative assumption since it amounts to a total neglect of the actual flexural flexibility within the elements, and it implies the extreme of a series of rigid elements that attain the out-of-plane slab deflection by means of exaggerated rotations restricted to the postulated crack locations. The crack width then follows from the product of the rotation times the radius to the center of rotation which corresponds to the neutral axis of the section. The approach is schematically represented in Figure 3.1, and is further described in the sample calculation per Appendix A. The resultant crack calculation is thus a reliable evaluation based on the fulfillment of compatibility with the analytically well defined flexural deformation of walls under pressure loading.

The crack widths calculated from rotational deformation are typically higher than those obtained per the stress-dependent formulation, and the final value is adjusted upwards by adding the axial tension component as previously calculated from the stress dependent formulation.

The most severe cracking develops at the base of the walls. At this location a single concentrated crack is a credible occurrence because of the pre-existing "cold" construction joint which, on the other hand, is appropriately safeguarded by a flexible waterstop. The flexible waterstop used is a proven product that has an excellent performance record during the many years of service in the industry. In the case under consideration, the watertight reliability of the waterstop is more decisively established upon considering the favorable confined and anchored conditions afforded to the waterstop by the reinforcing bars across the joint and the shear key provided. In addition, the reliability of waterstops subject to a much more severe function involving high differential motion across a wide 3-1/2 inch separation between two bodies of concrete has been reported favorably in reference (5).

REFERENCES

(1) Control of Cracking in Concrete Structures, reported by ACI Committee 224.

(2) "Nuclear Reactors and Earthquake" TID 7024 U.S. Atomic Energy Commission, August 1963.

(3) Nawy, E.G., "Crack Control Through Reinforcement Distribution in Two-Way Acting Slabs and Plates", ACI Journal, Proceedings, V. 69, No. 4, April 1972.

(4) Nawy, E. G., "Crack Control, Serviceability, and Limit Design of Two-Way Action Slabs and Plates", Engineering Research Bulletin No. 53, Rutgers University, 1972.

(5) Report of Test on Seismic Vibration and Water Leakage Test on Rubber Waterstops for use in Concrete Joint Systems, for W. R. Grace by Acton Testing Corporation, February 1974.

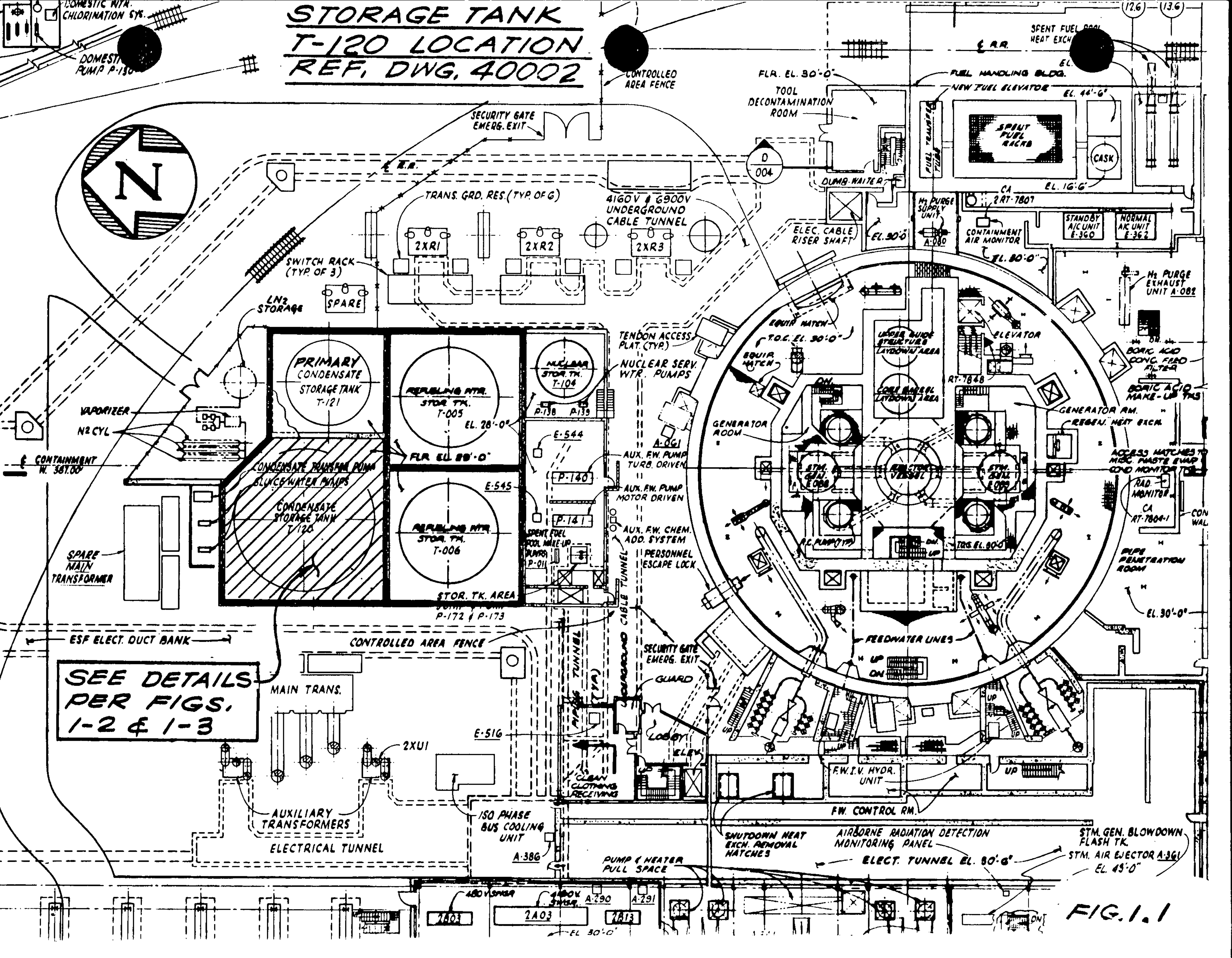
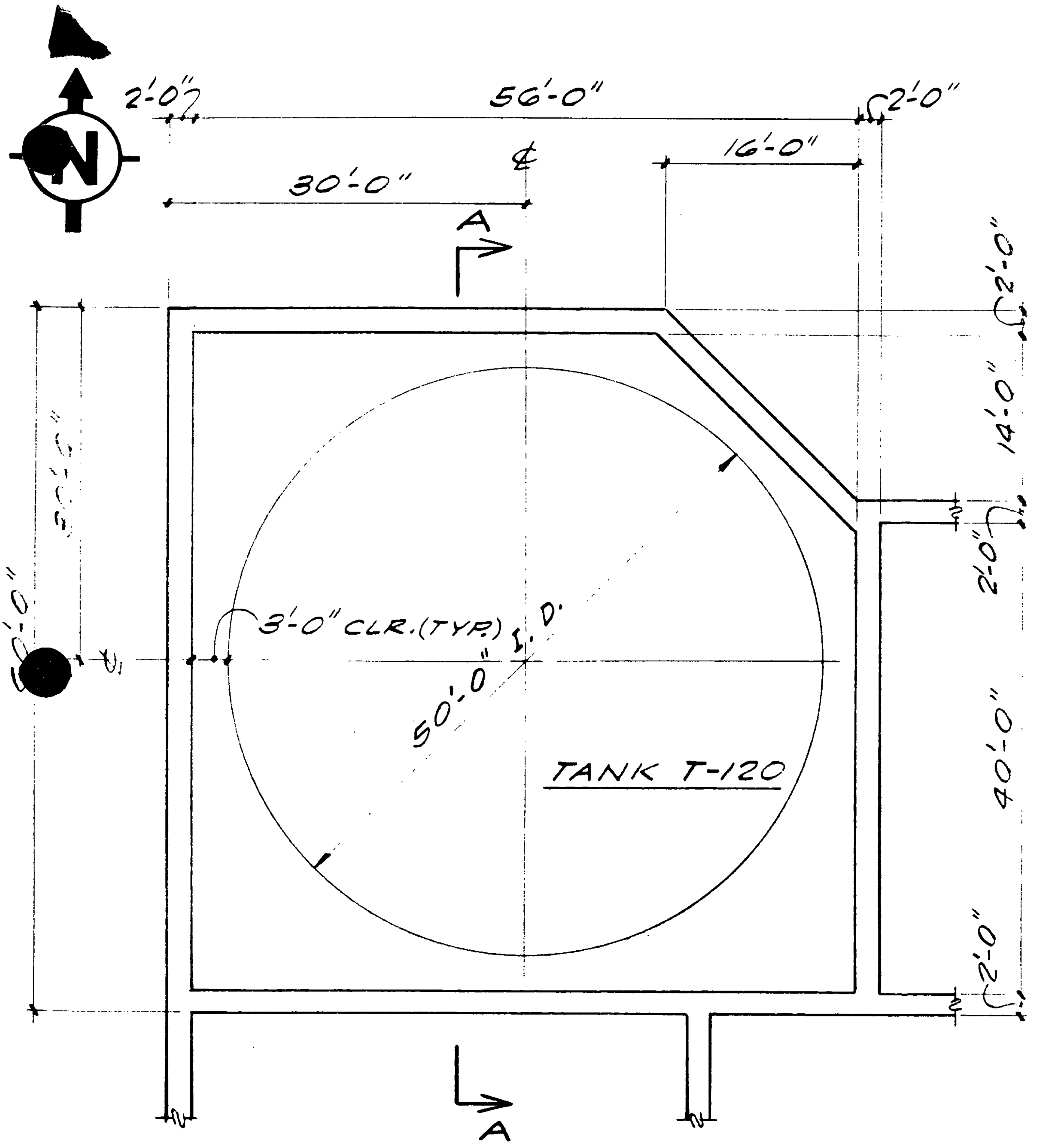
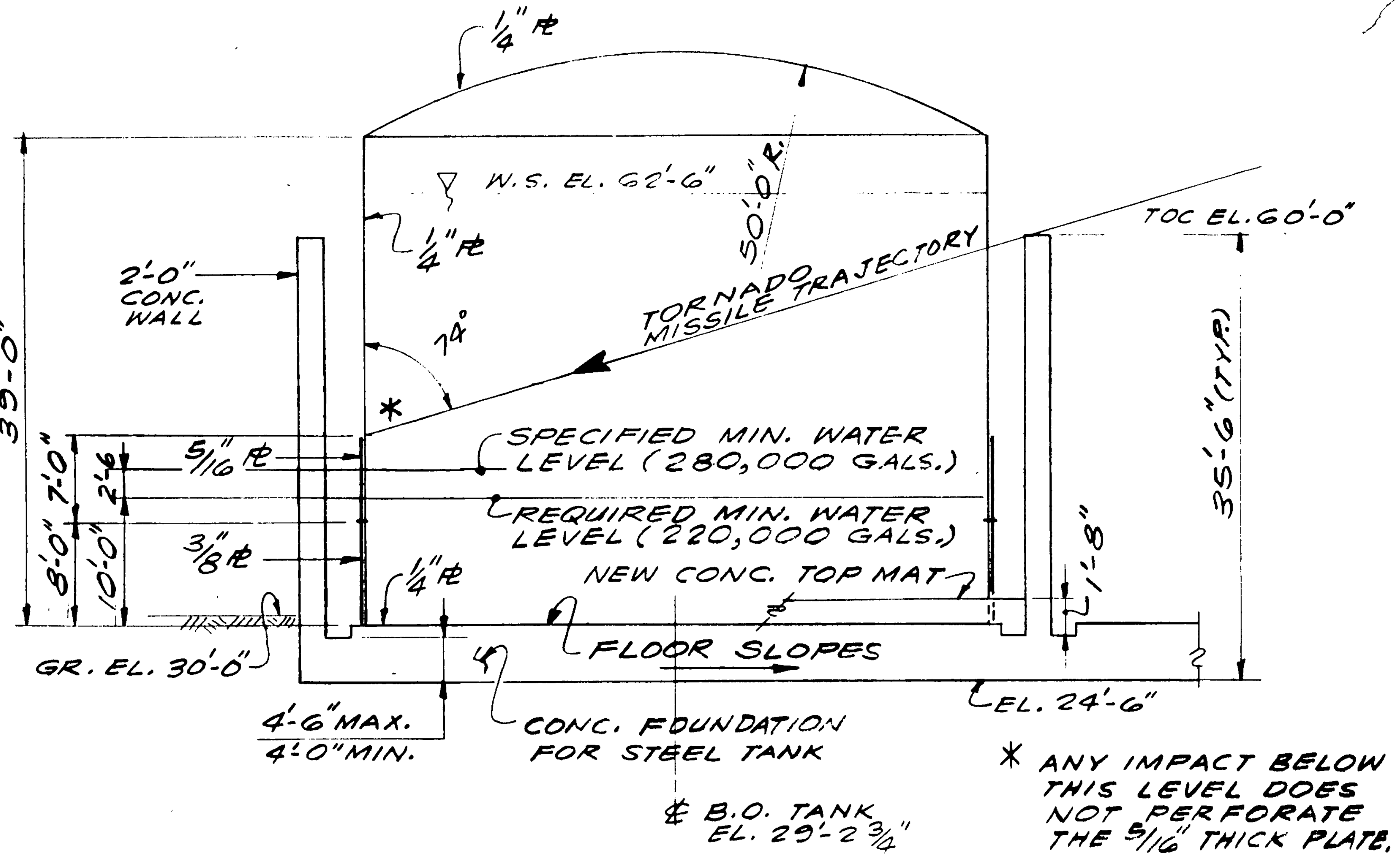

STORAGE TANK
T-120 LOCATION
REF. DWG. 40002
CONTROLLED AREA FENCE
SECURITY GATE EMERG. EXIT
TRANS. GRD. RES. (TYP. OF 6)
4160V & 6900V UNDERGROUND CABLE TUNNEL
2XR1
2XR2
2XR3
SWITCH RACK (TYP. OF 3)
SPARE
LN2 STORAGE
VAPORIZER
N2 CYL
CONTAINMENT W. 367.00'
PRIMARY CONDENSATE STORAGE TANK T-121
REFUELING WTR. STOR. TK. T-005
NUCLEAR STOR. TK. T-104
CONDENSATE STORAGE TANK T-120
REFUELING WTR. STOR. TK. T-006
FLR. EL. 28'-0"
EL. 28'-0"
E-544
E-545
P-140
P-141
P-138
P-139
NUCLEAR SERV. WTR. PUMPS
TENDON ACCESS PLAT. (TYP.)
AUX. F.W. PUMP TURB. DRIVEN
AUX. F.W. PUMP MOTOR DRIVEN
AUX. F.W. CHEM. ADD. SYSTEM
PERSONNEL ESCAPE LOCK
STOR. TK. AREA
P-172 & P-173
SPARE MAIN TRANSFORMER
ESF ELECT. DUCT BANK
CONTROLLED AREA FENCE
SEE DETAILS PER FIGS. 1-2 & 1-3
MAIN TRANS.
2XU1
E-516
AUXILIARY TRANSFORMERS
ELECTRICAL TUNNEL
ISO PHASE BUS COOLING UNIT
A-386
UNDERGROUND CABLE TUNNEL
SECURITY GATE EMERG. EXIT
GUARD
LOBBY
ELEV.
CLEAN CLOTHING RECEIVING
PUMP & HEATER PULL SPACE
480V SWGR
4160V SWGR
A-290
A-291
2A03
2B13
EL. 30'-0"
SHUTDOWN HEAT EXCH. REMOVAL HATCHES
AIRBORNE RADIATION DETECTION MONITORING PANEL
ELECT. TUNNEL EL. 30'-6"
F.W. CONTROL RM.
F.W.I.V. HYDR. UNIT
STM. GEN. BLOWDOWN FLASH TK.
STM. AIR EJECTOR A-361
EL. 45'-0"
FEEDWATER LINES
UP
DN
EL. 30'-0"
PIPE PENETRATION ROOM
GENERATOR ROOM
GENERATOR RM.
REGEN. HEAT EXCH.
RAD. MONITOR
CA RT-7804-1
ACCESS HATCHES TO MISC. WASTE EVAP. COND. MONITOR TKS.
BORIC ACID MAKE-UP TKS
BORIC ACID CONC. FEED FILTER
H2 PURGE EXHAUST UNIT A-082
ELEVATOR
RT-7848
UPPER GUIDE STRUCTURE LAYDOWN AREA
CORE BARREL LAYDOWN AREA
REACTOR VESSEL
STM. GEN. E-088
STM. GEN. E-089
R.C. PUMP (TYP.)
TOC EL. 30'-0"
EQUIP. HATCH
T.O.C. EL. 30'-0"
A-061
ELEC. CABLE RISER SHAFT
EL. 30'-0"
H2 PURGE SUPPLY UNIT
A-080
CONTAINMENT AIR MONITOR
CA 2RT-7807
STANDBY A/C UNIT E-360
NORMAL A/C UNIT E-362
TOOL DECONTAMINATION ROOM
FLR. EL. 30'-0"
FUEL HANDLING BLDG.
NEW FUEL ELEVATOR
EL. 44'-6"
SPENT FUEL RACKS
CASK
EL. 16'-6"
FUEL TRANSFER TUBE
DUMB WAITER
D 004
SPENT FUEL POOL HEAT EXCH.
12.6
13.6
DOMESTIC WTR. CHLORINATION SYS.
DOMESTIC PUMP P-150
SPENT FUEL POOL MAKE-UP PUMPS P-011
N

FIG. 1.1



PLAN

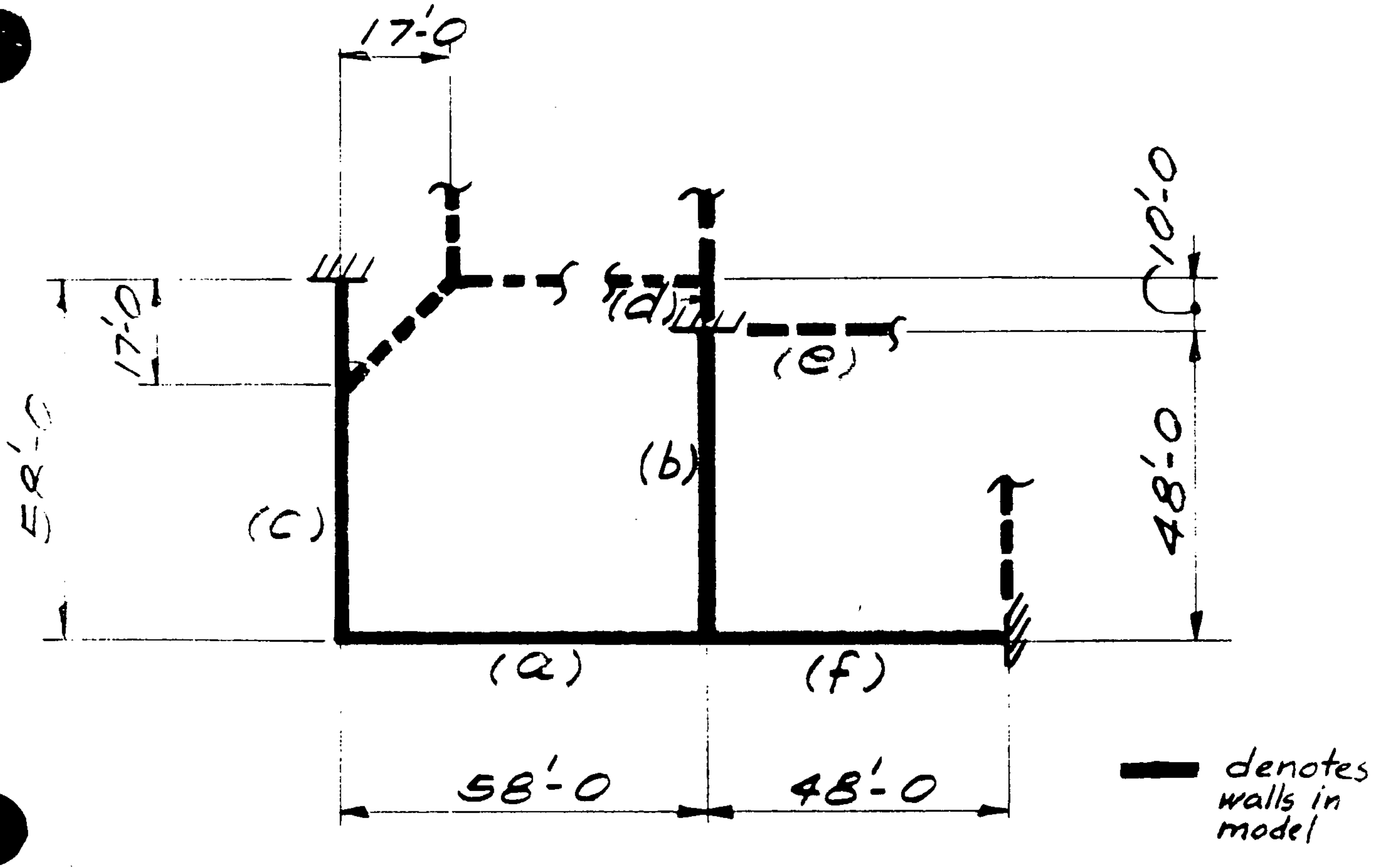
FIGURE 1.2



* ANY IMPACT BELOW THIS LEVEL DOES NOT PERFORATE THE 5/16" THICK PLATE.

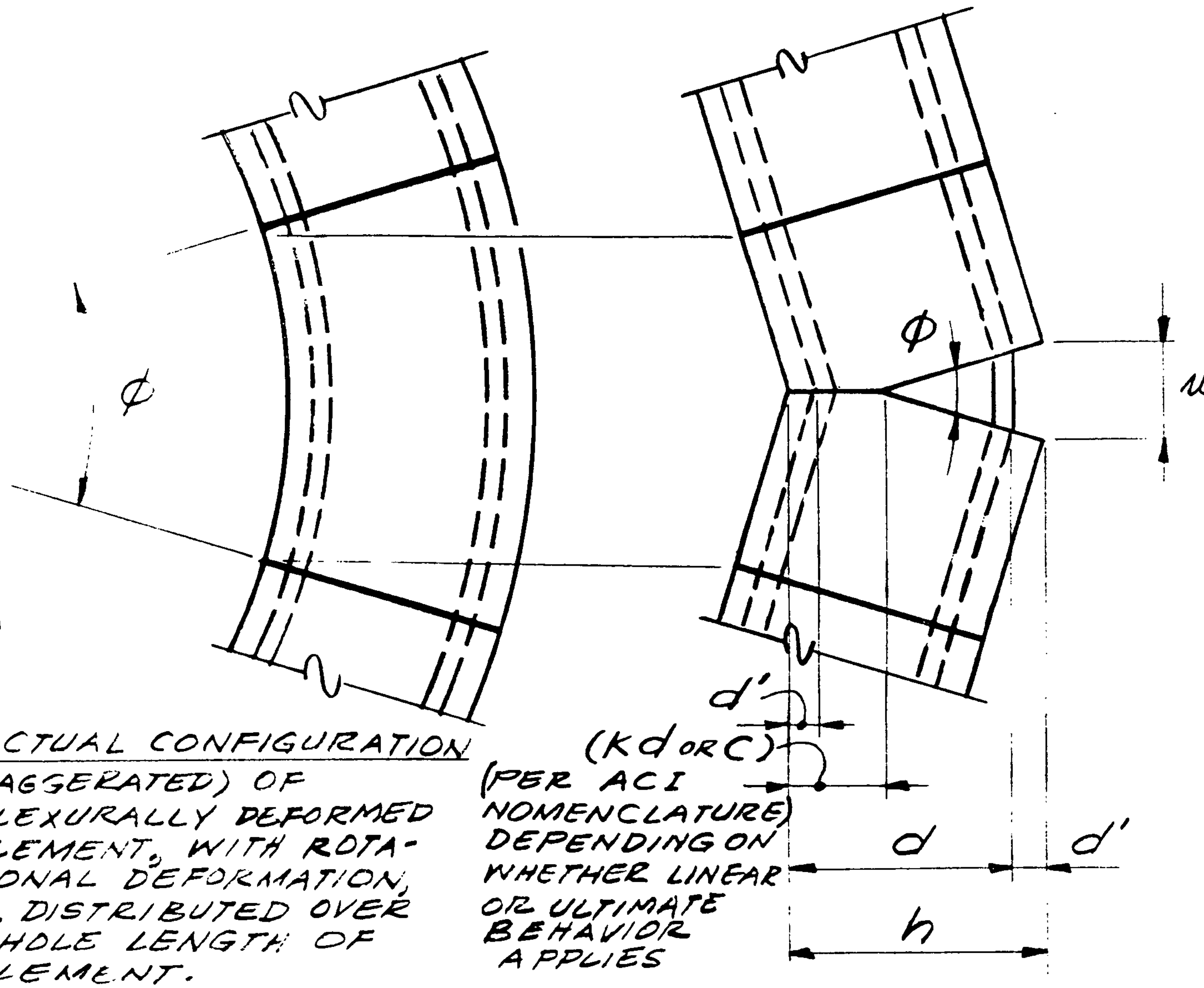
SECTION A-A

FIGURE 1.3



PLAN OF WALLS AND BOUNDARY CONDITIONS FOR FINITE ELEMENT MODEL.

FIGURE 2.1



ACTUAL CONFIGURATION (EXAGGERATED) OF FLEXURALLY DEFORMED ELEMENT, WITH ROTATIONAL DEFORMATION, ϕ, DISTRIBUTED OVER WHOLE LENGTH OF ELEMENT.

CONSERVATIVE IDEALIZATION OF FLEXURALLY DEFORMED ELEMENT FOR CALCULATION OF CRACK WIDTH, w.

$w = \phi\,(h - kd)$ FOR LINEAR CASE

$w = \phi\,(h - c)$ FOR ULTIMATE CASE

FIGURE 3.1

4.0 Technical Approach and Basis for Leakage Analysis

The leakage calculation is based on theoretical expressions for flow through parallel plates extended to model flow through cracks in the reinforced concrete walls. A summary of the deriviation of the equations is outlined on Table 4.1. The application of equation (7) of Table 4.1 requires that the actual crack pattern be represented by horizontal cracks. However, the actual cracks are of complex cross section and of variable aperture, therefore, equivalent constant apertures of cracks are calculated by the relationships summarized on Table 4.2.[4]. The vertical cracks were modeled as equally dimensioned horizontal cracks at elevations 16.7, 11, and 6.2 ft. For example, the 5.8 ft. segment of each vertical crack from the water surface (elevation 22.5 ft.) to elevation 16.7 ft. was taken as a 5.8 ft. long horizontal crack at elevation 16.7 ft. Similarly, the segment of each vertical crack between elevation 16.7 and 11 ft. was taken as a 5.7 ft. long crack at elevation 11 ft., etc. By considering the equivalent horizontal cracks as located at the lowest level of each vertical segment larger applicable heads result, and a conservatively larger flow quantity is thereby calculated. Use of this conservative procedure considerably simplified the calculations while maintaining the conservatism. The leakage in the room can then be calculated in increments using equation (7) of Table 4.1.

4.1 Discussion of the Basis of and Conservatisms Inherent to the Equation Used to Estimate Leakage

The parallel plate model for crack flow has been applied by various researchers in the petroleum and water resources fields since the 1950's. A demonstration of the validity of the theory for laminar flow in both smooth and rough walled plates may be found, for example, in the work of Haitt[1], who showed that such flow obeys Darcy's law for low Reynolds numbers. This work was amplified and extended by Louis[3] who experimented on a wider range of surface roughness and flow rates, and produced a set of empirical equations for smooth and roughwalled cracks under both turbulent and laminar flow.

Louis' equations for smoothwalled and laminar flow are the same as those used in deriving equation (7). His equation for rough walled cracks is a function of the relative roughness $K_r = E/2b$, where E is defined as the depth of surface roughness (measured peak to trough), and b is the nominal crack aperture. If the crack exhibits surface roughness, the smooth-walled flow rate for a given gradient is reduced by the factor $1/(1 + 8.8\ K_r^{1.5})$. As an example, cracks with surface roughness equal to 20 percent of aperture, a value generally exceeded in concrete flexural cracks, would conduct approximately 22 percent less flow than smooth cracks. The calculated leakage determined herein is thus conservative since surface roughness effects were not considered.

Another factor which conservatively is not considered in the idealization to parallel plate flow is tortuosity, as it affects crack length. Flow rate for a given gradient is inversely proportional to crack length through the wall, thus the effect of crack tortuosity is to increase the actual length of the flow path and decrease the flow rate.

Turbulent flow was found by Louis(3) to occur when the Reynolds number exceeds about 1000 in rough-walled cracks or about 2300 in smooth-walled cracks. Under laminar conditions, the flow rate varies directly with the change in gradient, whereas under turbulent conditions, the flow rate varies approximately with the square root of the gradient. From the above conditions, it can be concluded that the use of a laminar smooth-walled parallel plate flow model for cracks will tend to over estimate the true flow rate which would occur in non-ideal rough-walled, tortuous fractures which may, in fact, experience turbulent flow. Therefore, the use of equation (7) will yield conservative estimates of flow through cracks in the reinforced concrete walls.

4.2 Cases Studies

The results of the analysis of the distribution of cracks and maximum crack openings are shown on Figures 4.2, 4.3, and 4.4. Specifically, Figure 4.2 shows the maximum crack pattern in the most critical wall due to water and seismic induced loadings for the condition of the concrete enclosure containing 500,000 gallons of water, and Figure 4.3 shows the corresponding crack pattern for 280,000 gallons of water in the enclosure. The crack pattern for the other three walls was calculated to be as shown on Figure 4.4 for the 500,000 gallon case. Equation (7) of Table 4.1 was used to estimate leakage through the cracks. All cracks were assumed to have a crosssectional configuration defined by the maximum aperture at the inside or outside face of the wall as defined on Figure 4.2 and 4.3, and to have a uniform reduction over a length of about 18 inches to a minimum aperture of 0.001 inches. This minimum aperture is constant over the last six inches of wall. In the case where the aperture at the inside face of the wall was less than .001 inches, a constant aperture equal to the aperture at the inside face of the wall was assumed. Equivalent uniform apertures were calculated for each crack using the relationships shown on Table 4.2. Vertical cracks were characterized in a conservative manner by two equivalent horizontal cracks at elevations 16.7, 11, and 6.2 ft. as described above.

The most severe crack occurs at the base of the wall where imposed seismic and hydrodynamic moments are the greatest. Figure 4.5 shows the most conservative interpretation of this crack considering the maximum apertures at the inside and outside faces of the wall simultaneously. The maximum crack aperture of 0.039 inches is shown on the inside face of the wall, and it decreases uniformly to a minimum aperture of .023 inches at the outside face of the wall. Because of the geometry of the shear key, a minimum aperture of .008 and 0.007 inches occur at segments BC and ED of the crack as shown in Figure 4.5.

In this respect it is noted that the effect of the outward hydrostatic loading imposed by the spilled total contents of the tank is to enlarge the crack aperture on the inside face of the walls and to reduce the aperture on the outside face. For this analysis a conservative combination of apertures was selected wherein the maximum hydrostatic loadings were considered in calculating the enlarged inside aperture but were disregarded in reducing the outside aperture.

The maximum crack apertures shown on Figures 4.3, 4.4, and 4.5 were developed from the governing crack analysis described above. The minimum aperture of .001 inches for all cracks (except for the foregoing conservative interpretation for the base crack in the wall shown on Figures 4.2 and 4.5) was developed considering the 250 to 500 psi range of normal compressive stress calculated at the various sections from the flexural moment obtained under the sustained hydrostatic load as presented in Appendix A. The stress is relevant to evaluate the minimum crack aperture at locations where the linear behavior prevails and the flexural compression zone may experience prior cracking due to moment reversal caused by the lateral seismic inertial load from walls. The effect of this normal compressive stress is to induce contact and closure of the crack, which in turn tends to decrease the flow rate. The effect has been evaluated from laboratory tests performed on cracks in various natural materials. Work performed by Iwai[2] on basalt, granite and marble is used to develop the effect of stress on crack aperture as shown on Figure 4.1. An aperture of .001 inches represents a conservative interpretation of the referenced test data.

In the case where leakage was calculated across the specific crack shown in Figure 4.5, the presence of the water stop was neglected as were the waterproofing compound between the inside wall face and the new top mat slab. The presence of the new concrete mat itself, which will be placed to improve seismic anchorage of the base of the tank, was also neglected for conservatism.

Specific cases included in the analyses were:

1. the maximum crack pattern with no leakage through the bottom crack due to the presence of the water stop for an initial volume of 500,000 gallons of water (Figure 2);

2. the same as case 1 but conservatively neglecting the water stop throughout the total crack length and assuming

leakage through the bottom crack on the vertical wall characterized on Figure 4.2 excluding the cracks identified by asterisks. These crack widths become .001 inches upon consideration of closure due to hydrostatic loading. Case 5 describes the conservative case where hydrostatic closure was neglected.

3. the maximum crack pattern with no leakage through the bottom crack due to the presence of the water stop for an initial volume of 280,000 gallons of water;

4. the same as case 3 but neglecting the water stop and assuming leakage through the bottom crack;

5. the same as case 1 but neglecting the water stop in the most critical wall and assuming the most critical combination of apertures are indicated by the table in Figure 4.5; and

6. additional leakage obtained from permeable flow through sound concrete.

For all the above cases except cases 5 and 6, the crack pattern in the most critical wall was conservatively assumed to occur in all four walls by multiplying the leakage for the most critical wall (Figures 4.2 and 4.3) by a factor of 4. Under case 6 the seepage due to the permeability of uncracked concrete was considered. The permeability coefficient was determined per references (5) and (6) based on the maximum aggregate size of 1-1/2 inch and the favorably low water/cement ratio of 0.57 corresponding to the concrete mix (with 10% pozzolan) used in the enclosure structure. The total surface area of the walls and base mat was considered, and conservatively, all were subject to the maximum head of 22.5 feet of water.

At the request of the NRC staff a seventh case was also evaluated to determine how large a localized aperture at the bottom of the wall would have to be to discharge the 260,000 gallons of excess water not required for cooling over the 24 hour period upon starting with the 500,000 gallon initial volume case. The maximum length of opening was assumed to be equal to the reinforcing bar spacing

(1-ft) because the reinforcing steel would restrain a localized opening due to its tensile capacity. With this restraint in effect, the minimum aperture of the bottom crack was back calculated such that 260,000 gallons of water would be discharged in 24 hours. In the interest of conservatism all calculations were made assuming no reductions in head due to intentional drawdown for cooling water.

4.3 Results of Analyses

The calculated leakage for cases 1 and 2 defined above was less than 1500 and 3000 gallons in 24 hours, respectively. The corresponding leakage for cases 3 and 4 was less than 150 gallons for either case. For case 5, less than 40,000 gallons of leakage in 24 hours is calculated. For case 6 the calculated leakage is insignificant amounting to 1.5 gallons over the 24 hour period. For the localized 1/2- to 1-foot long crack at the bottom of the wall the average aperture through the wall required to cause 260,000 gallons of leakage in 24 hours was calculated to be about .05 to .08 inches. This is almost two orders of magnitude greater than the conservative .001-inch minimum aperture of most of the crack and about one order of magnitude greater than the minimum aperture of the base crack (where the water stop is effective) of the most critical wall characterized on Figure 4.2. If the reduction in head resulting from the intentional drawdown of the water in the room is incorporated in this last extreme analysis of a localized opening, an additional 80,000 to 100,000 gallons of water would remain in the room at the end of 24 hours (i.e., 80,000 to 100,000 gallons less leakage from cracks would occur).

In summary, the Applicant estimates that less than 1500 gallons of water would leak through the Condensate Storage Building walls in 24 hours under the worst loading conditions (i.e., high water level due to 500,000 gallons of water with DBE loading). By neglecting the effectiveness of the water stop throughout the length of the base crack in the most critical wall the resulting leakage would still be less than 3000 gallons which is less than 2% of the excess 260,000 gallons of water available for cooling for that period.

Assuming the most critical combination of apertures as indicated in the table on Figure 4.5 (a condition that is not credible because the maximum aperture on the outside face neglects the hydrostatic head but represents an extreme limiting calculation) leakage would be less than 40,000 gallons in 24 hours. Further, the Applicant calculates that a local aperture of a crack restrained in length by the reinforcing bar spacing could be almost two orders of magnitude greater than the conservative .001 inch opening at the outside face of the wall and still leave almost 100,000 gallons of excess water in the room after 24 hours.

REFERENCES

(1) Huitt, J. L., "Fluid flow in simulated fractures," AICHE Journal, v. 2, p. 259, 1956.

(2) Iwai, K., "Fundamental studies of fluid flow through a single fracture," Ph. D. thesis, University of California, Berkeley, 208 p., 1976.

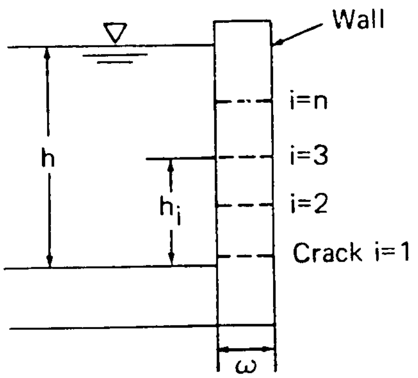
(3) Louis, C., "Stromungsvorgange in Kluftigen Medien and ihre Wirkung auf die Standischerheit von Bauwerken und Boschungen im Fells," Dissertation Universitat (TH) Karsruhe, 1967. Also published in English as: "A study of groundwater flow in jointed rock and its influence on the stability of rock masses," Imperial College Rock Mechanics Research Report No. 10, September 1969.

(4) Wilson, C. R., and Witherspoon, P. A., "An investigation of laminar flow in fractured porous rocks," Department of Civil Enginering Publication 70-6, University of California, Berkeley, 1970.

(5) Concrete Manual. A water resources technical publication, U.S. Department of the Interior, Bureau of Reclamation. Eighth Edition, 1975; Figure 17.

(6) Mass Concrete for Dams and Other Massive Structures, reported by ACI Committee 207; Table 3.5.1.

Table 4.1 SUMMARY DEVIATION OF LEAKAGE EQUATIONS



A = Area of Room
b_i = Aperture of Crack i
L_i = Length of Crack i
ρ = Mass Density of Water
g = Acceleration due to Gravity
μ = Viscosity of Water
ω = Width of Wall
h = Height of Water in Room

line no

FLOW RATE: $Q = K \; i \; A'$ — K: Hydraulic Conductivity; i: Gradient; A': Flow Area (1)

USING THE PARALLEL PLATE MODEL (HUITT 1956)

$$A \frac{dh}{dt} = \left(\frac{b^2 \rho g}{12 \mu}\right) \left(\frac{h - h_i}{\omega}\right) \left(bL\right) \qquad (2)$$

FOR n CRACKS:

$$A \frac{dh}{dt} = -\left\{ \left[\sum_{i=1}^{n} \frac{\rho g}{\mu} \frac{b_i^3}{12} \frac{L_i}{\omega} \right] h(t) - \sum_{i=1}^{n} \frac{\rho g}{\mu} \frac{b_i^3}{12} \frac{L_i}{\omega} h_i \right\} \qquad (3)$$

ISOLATING CONSTANTS

$$\text{let } C_i = \frac{\rho g}{\mu} \frac{b_i^3}{12} \frac{L_i}{\omega} \quad \text{and} \quad C_n = \sum_{i=1}^{n} C_i \qquad (4)$$

$$\text{also } M_i = C_i h_i \quad \text{and} \quad M_n = \sum_{i=1}^{n} M_i \qquad (5)$$

AND SIMPLIFYING:

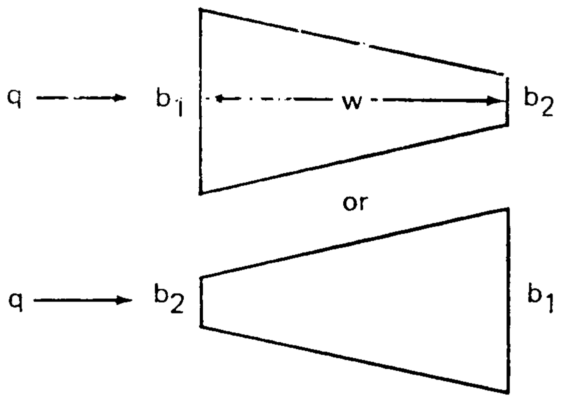
$$A \frac{dh}{dt} = -\left[C_n h - M_n \right] \qquad (6)$$

UPON INTEGRATION:

$$-\frac{A}{Cn} \ln \left[\frac{C_n h_t - M_n}{C_n h_o - M_n} \right] = t - t_o \qquad (7)$$

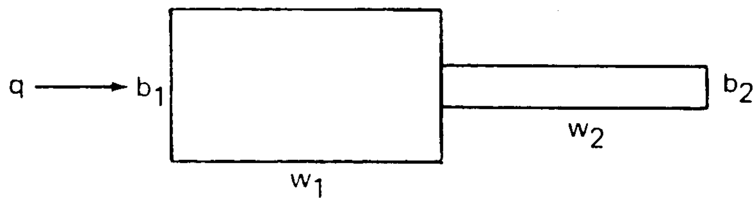
Table 4.2 CALCULATION OF EQUIVALENT APERTURES

Wedge shaped cracks: $b_1 > b_2$



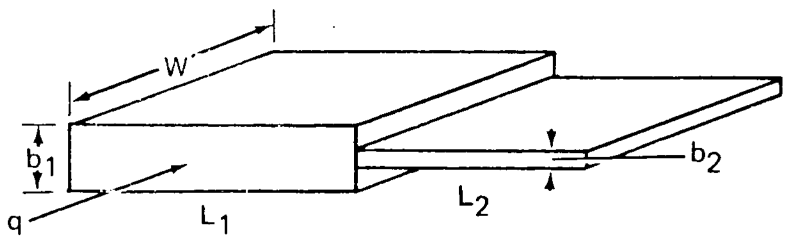
$$b^3_{eff} = \frac{2b_1^2\, b_2^2}{b_1 + b_2}$$

Cracks in series:



$$b^3_{eff} = \frac{w_1 + w_2}{\dfrac{w_1}{b_1^3} + \dfrac{w_2}{b_2^3}}$$

Cracks in parallel:



$$b^3_{eff} = \frac{L_1\, b_1^3 + L_2\, b_2^3}{L_1 + L_2}$$

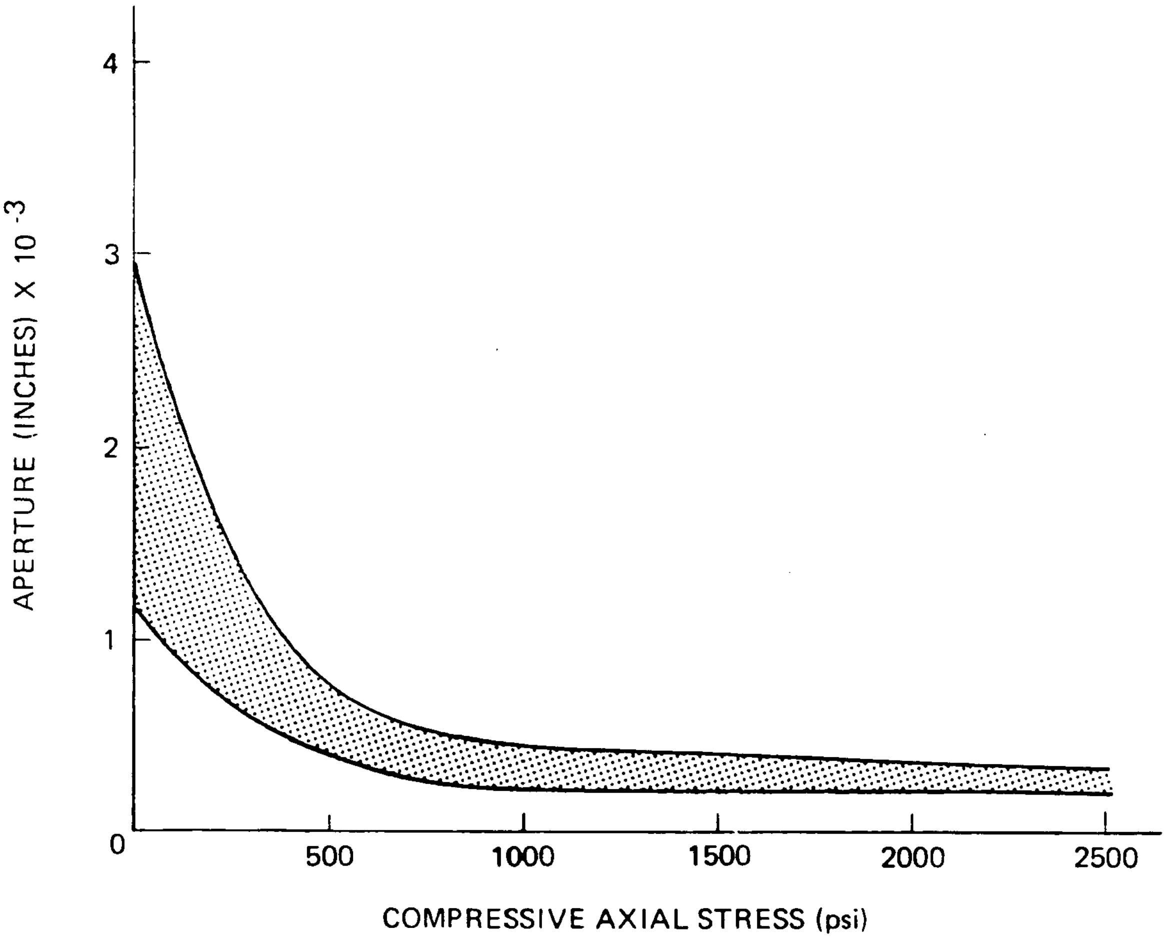


Figure 4.1 EFFECT OF COMPRESSIVE STRESS ON APERTURE

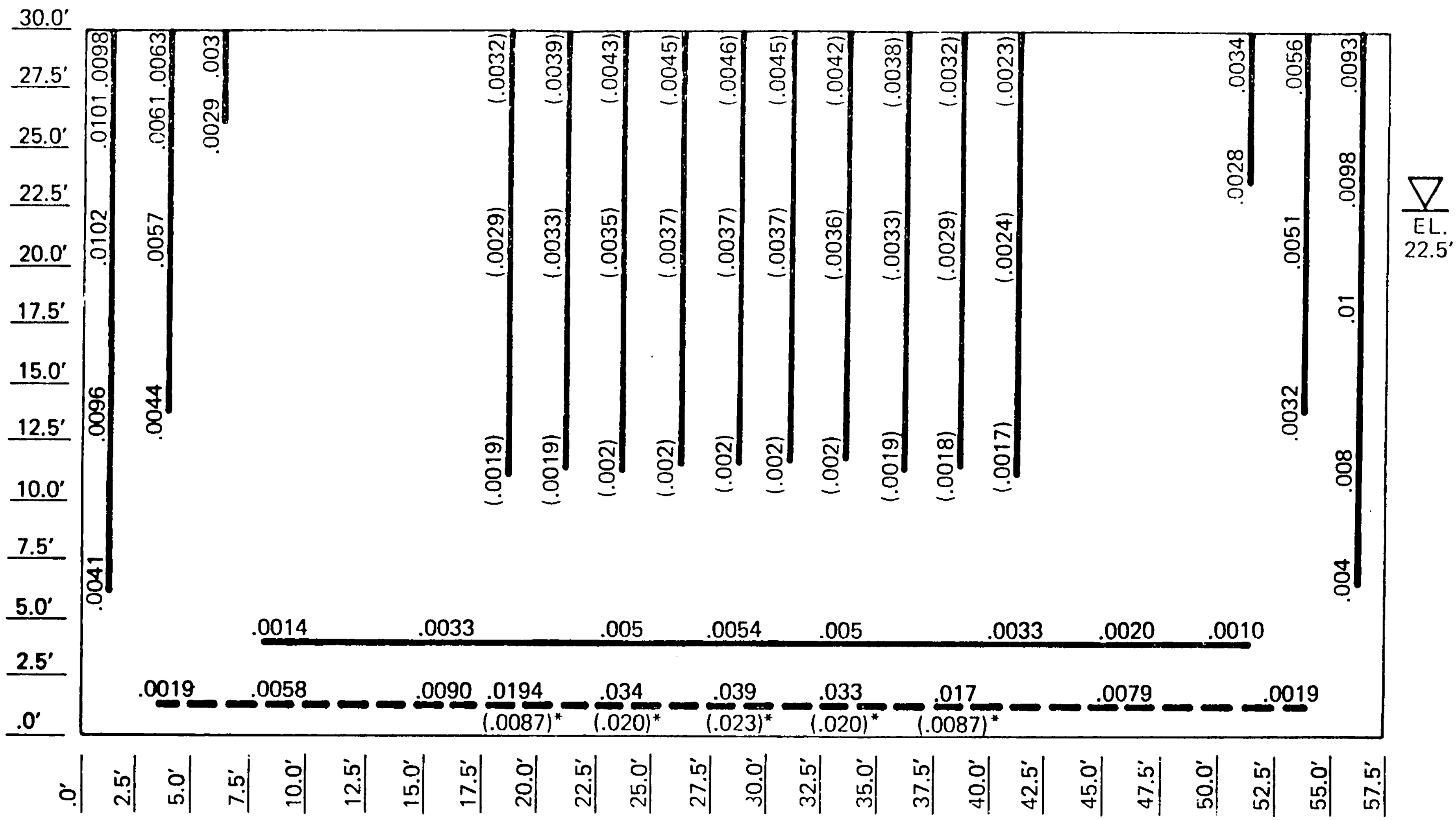
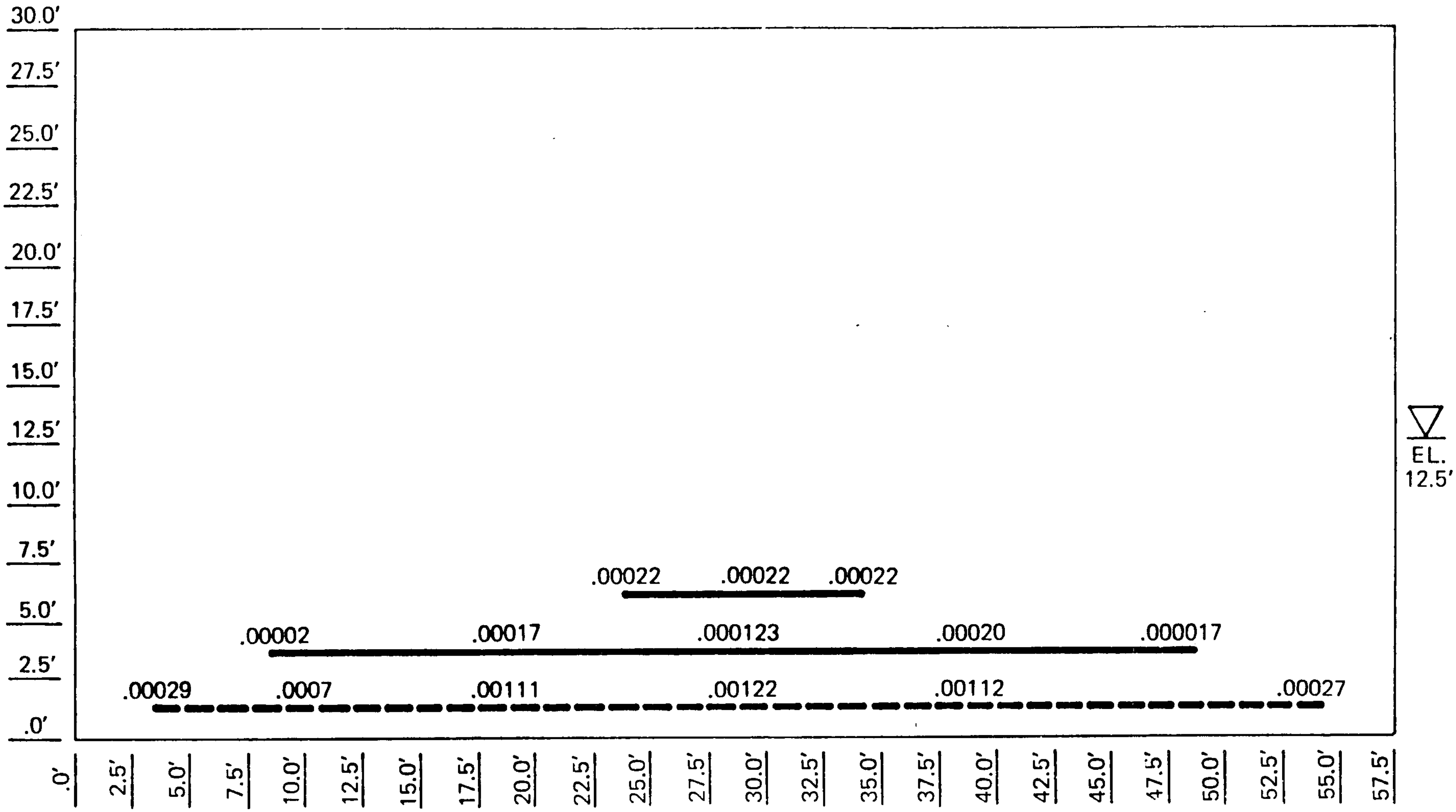


Figure 4.2 CONDENSATE STORAGE TANK AREA WALL (a)
DIAGRAM SHOWING CRACK APERTURES IN INCHES

Crack apertures shown relate to maximum aperture on inside wall face.
Crack apertures shown in parenthesis indicate maximum aperture on outside wall face.
Bottom crack shown by a dashed line contains the water stop.
* These apertures occur only with no hy[illegible]atic pressures on the wall; therefore the combination of apertures shown is [illegible] most conservative combination.



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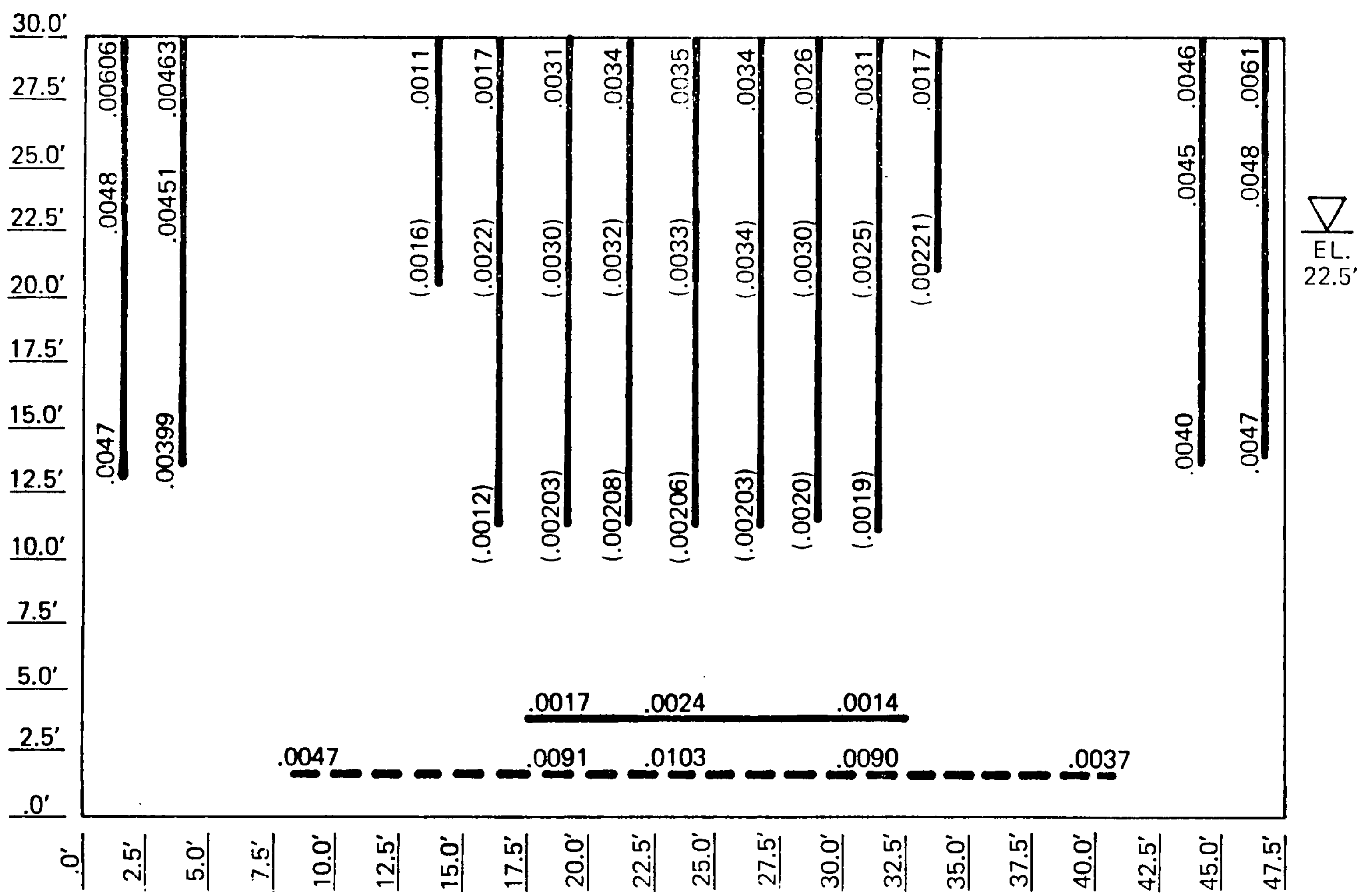
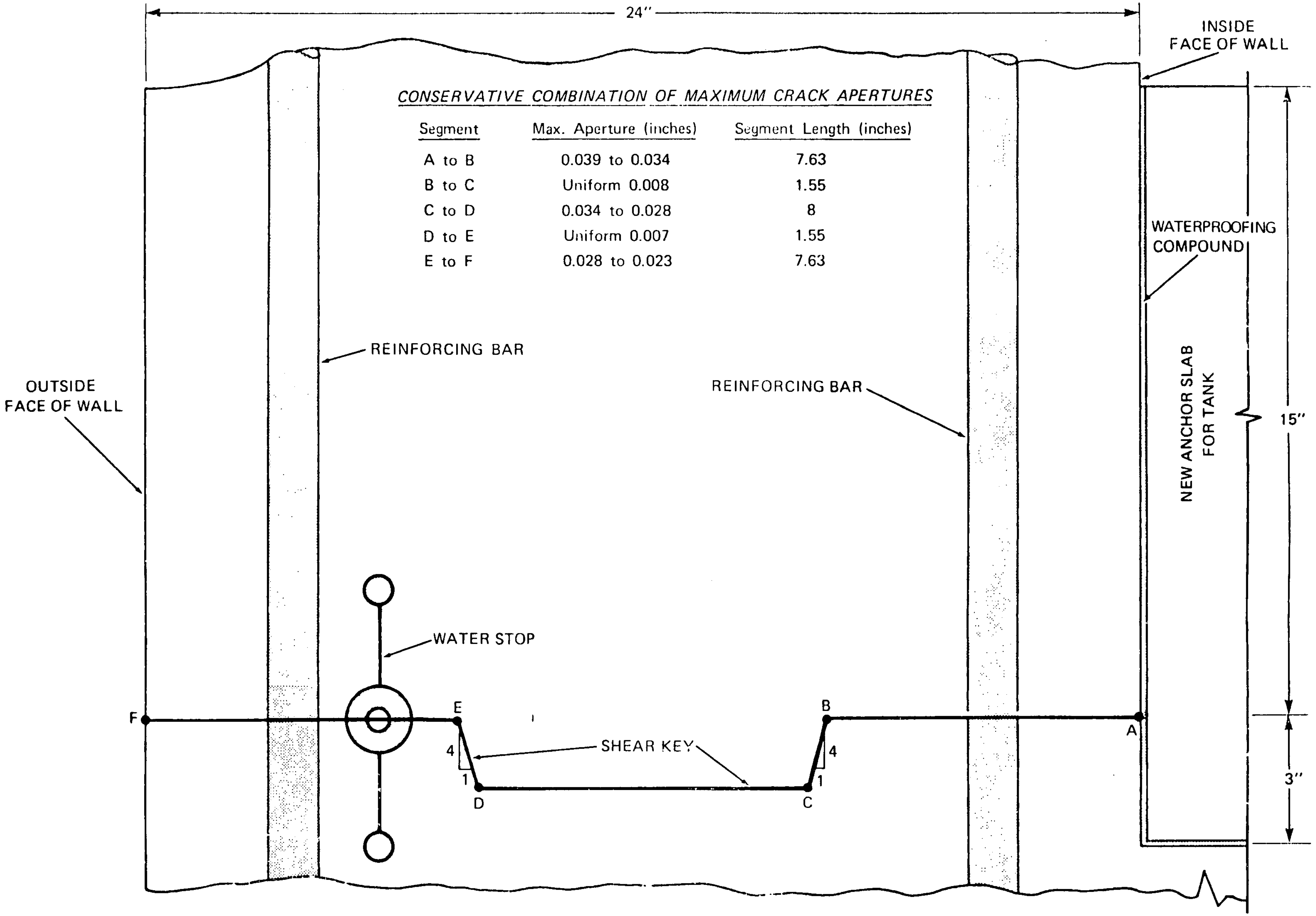


Figure 4.4 CONDENSATE STORAGE TANK AREA WALL (b)
DIAGRAM SHOWING CRACK APERTURES IN INCHES

Crack apertures shown relate to maximum aperture on inside wall face.
Crack apertures shown in parenthesis indicate maximum aperture on outside wall face.
Bottom crack shown by a dashed line contains the water stop.



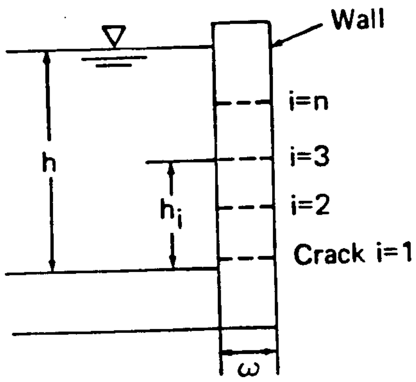
CONSERVATIVE COMBINATION OF MAXIMUM CRACK APERTURES

Segment	Max. Aperture (inches)	Segment Length (inches)
A to B	0.039 to 0.034	7.63
B to C	Uniform 0.008	1.55
C to D	0.034 to 0.028	8
D to E	Uniform 0.007	1.55
E to F	0.028 to 0.023	7.63

SCALE [illegible] = 3"

Figure 4.5 SECTION THROUGH BASE OF WALL

Table 4.1 SUMMARY DEVIATION OF LEAKAGE EQUATIONS



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μ = Viscosity of Water
ω = Width of Wall
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line no

FLOW RATE: $$Q = \underbrace{K}_{\text{Hydraulic Conductivity}} \quad \underbrace{i}_{\text{Gradient}} \quad \underbrace{A'}_{\text{Flow Area}}$$ (1)

USING THE PARALLEL PLATE MODEL (HUITT 1956) $$A\frac{dh}{dt} = \left(\frac{b^2\rho g}{12\mu}\right)\left(\frac{h - h_i}{\omega}\right)\left(bL\right)$$ (2)

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ISOLATING CONSTANTS

let $C_i = \frac{\rho g}{\mu}\frac{b_i^3}{12}\frac{L_i}{\omega}$ and $C_n = \sum_{i=1}^{n} C_i$ (4)

also $M_i = C_i h_i$ and $M_n = \sum_{i=1}^{n} M_i$ (5)

AND SIMPLIFYING: $$A\frac{dh}{dt} = -\left[C_n h - M_n\right]$$ (6)

UPON INTEGRATION: $$-\frac{A}{Cn}\ln\left[\frac{C_n h_t - M_n}{C_n h_o - M_n}\right] = t - t_o$$ (7)

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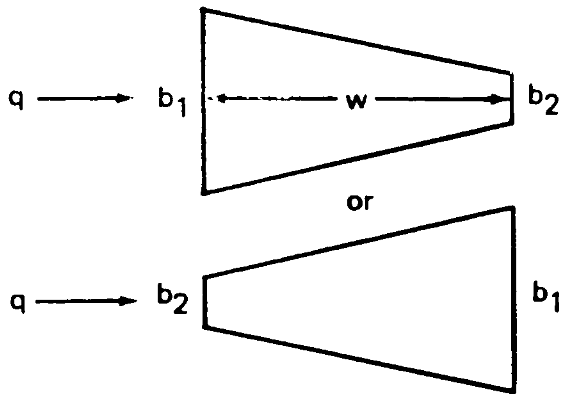
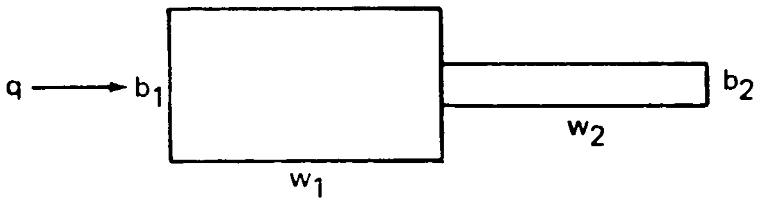


$$b^3_{eff} = \frac{2b_1^2\, b_2^2}{b_1 + b_2}$$

Cracks in series:



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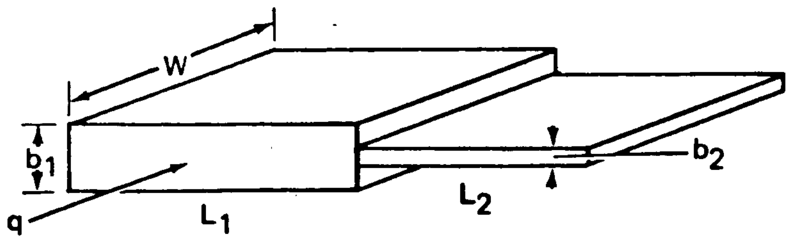
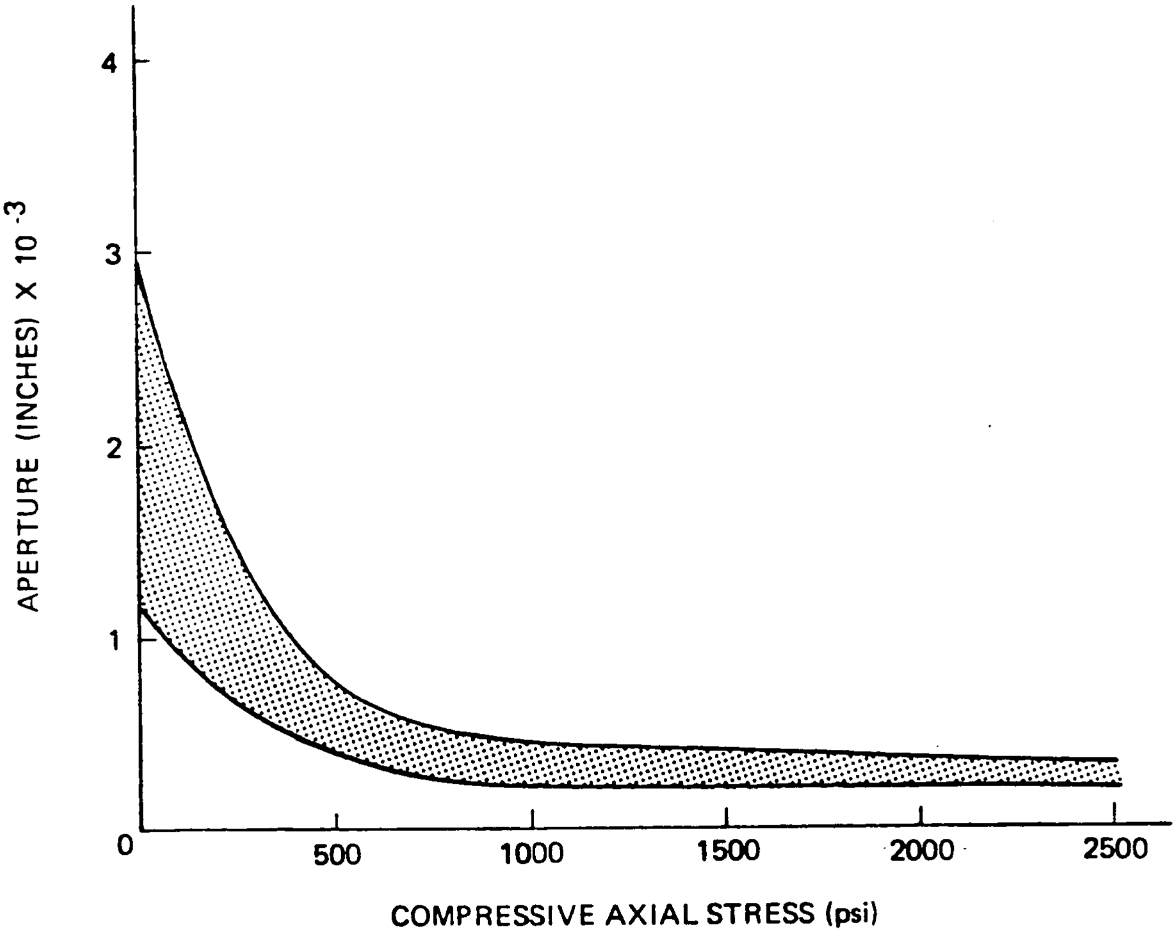
Cracks in parallel:



$$b^3_{eff} = \frac{L_1\, b_1^3 + L_2\, b_2^3}{L_1 + L_2}$$



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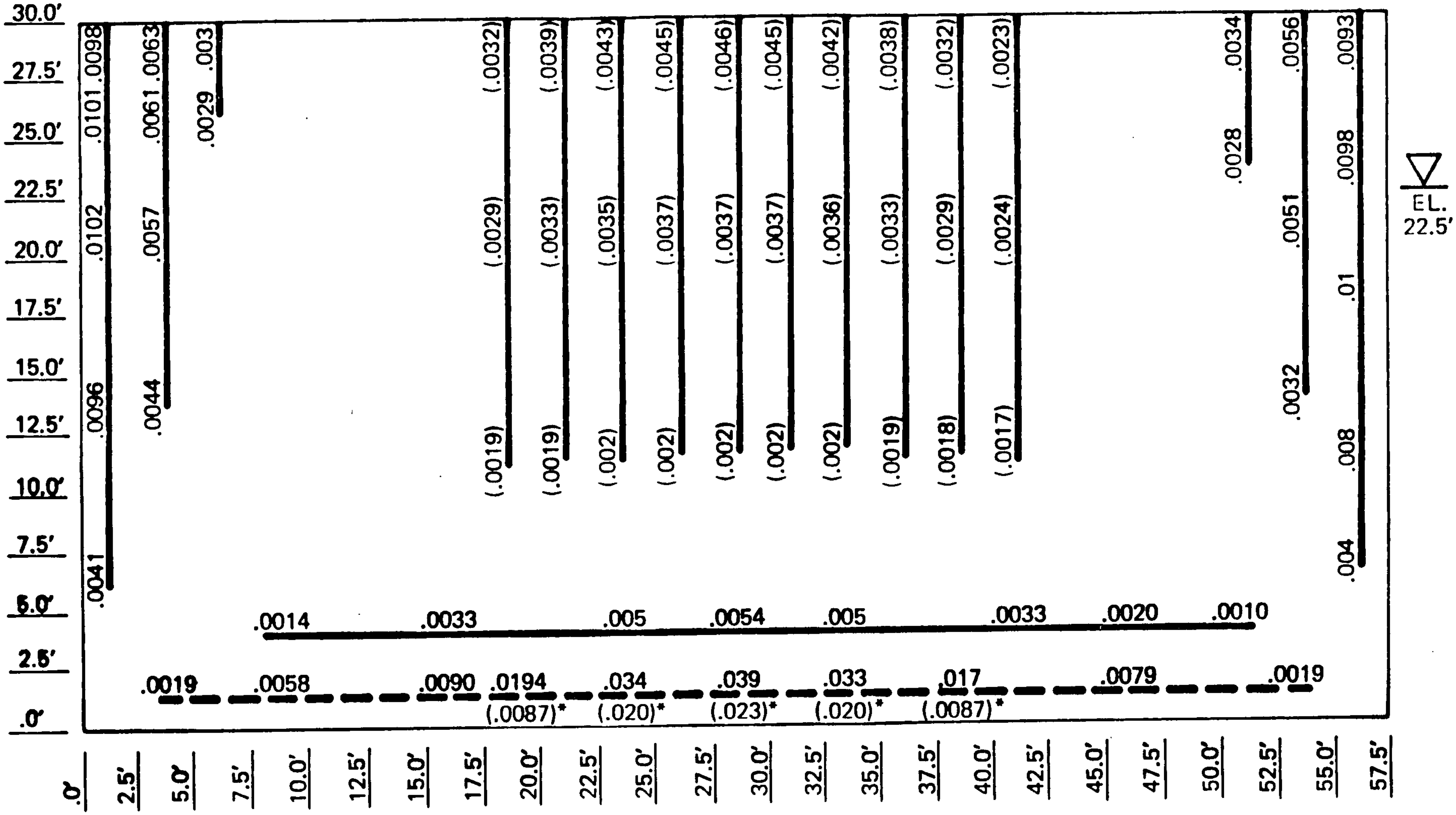

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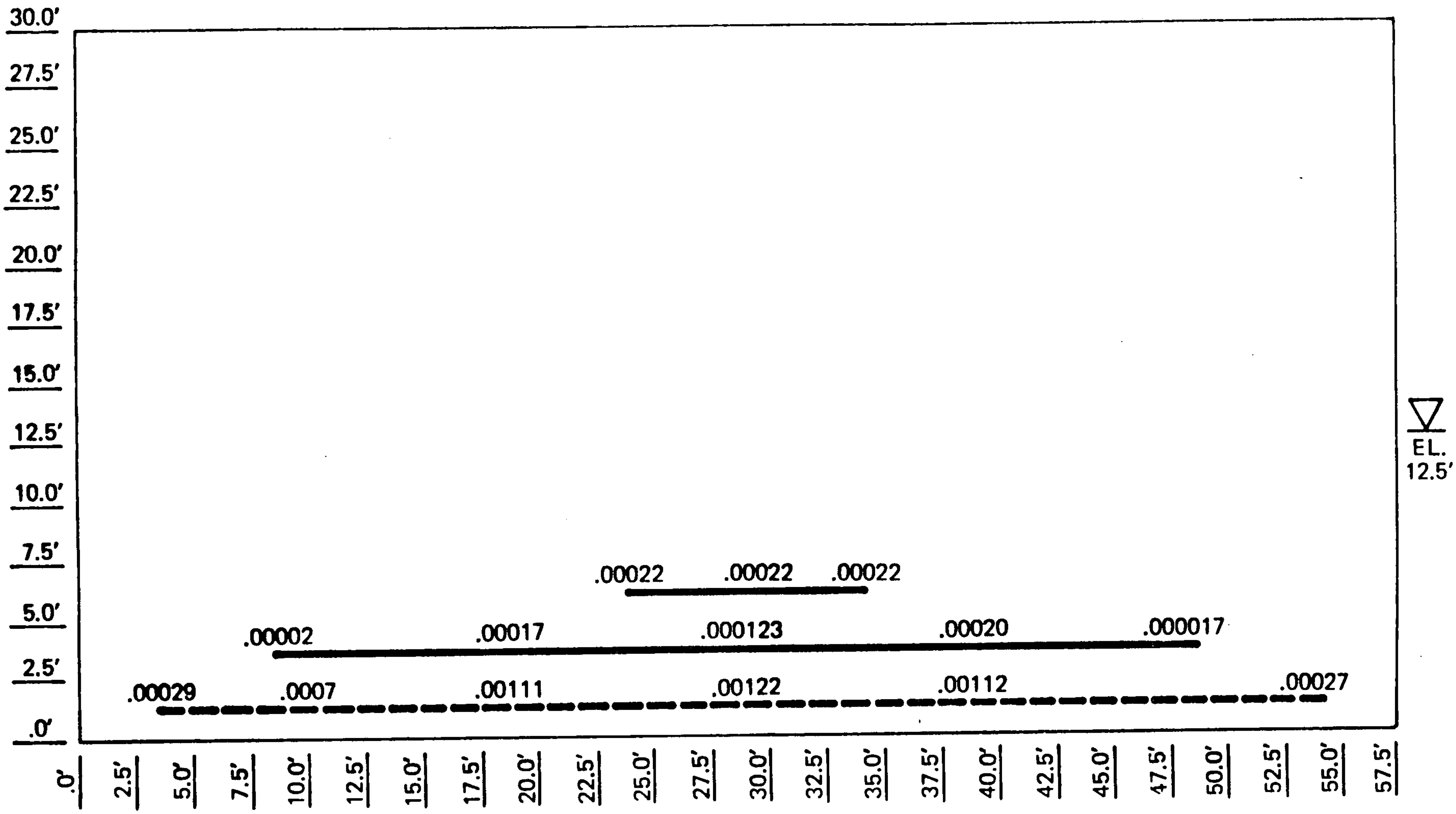

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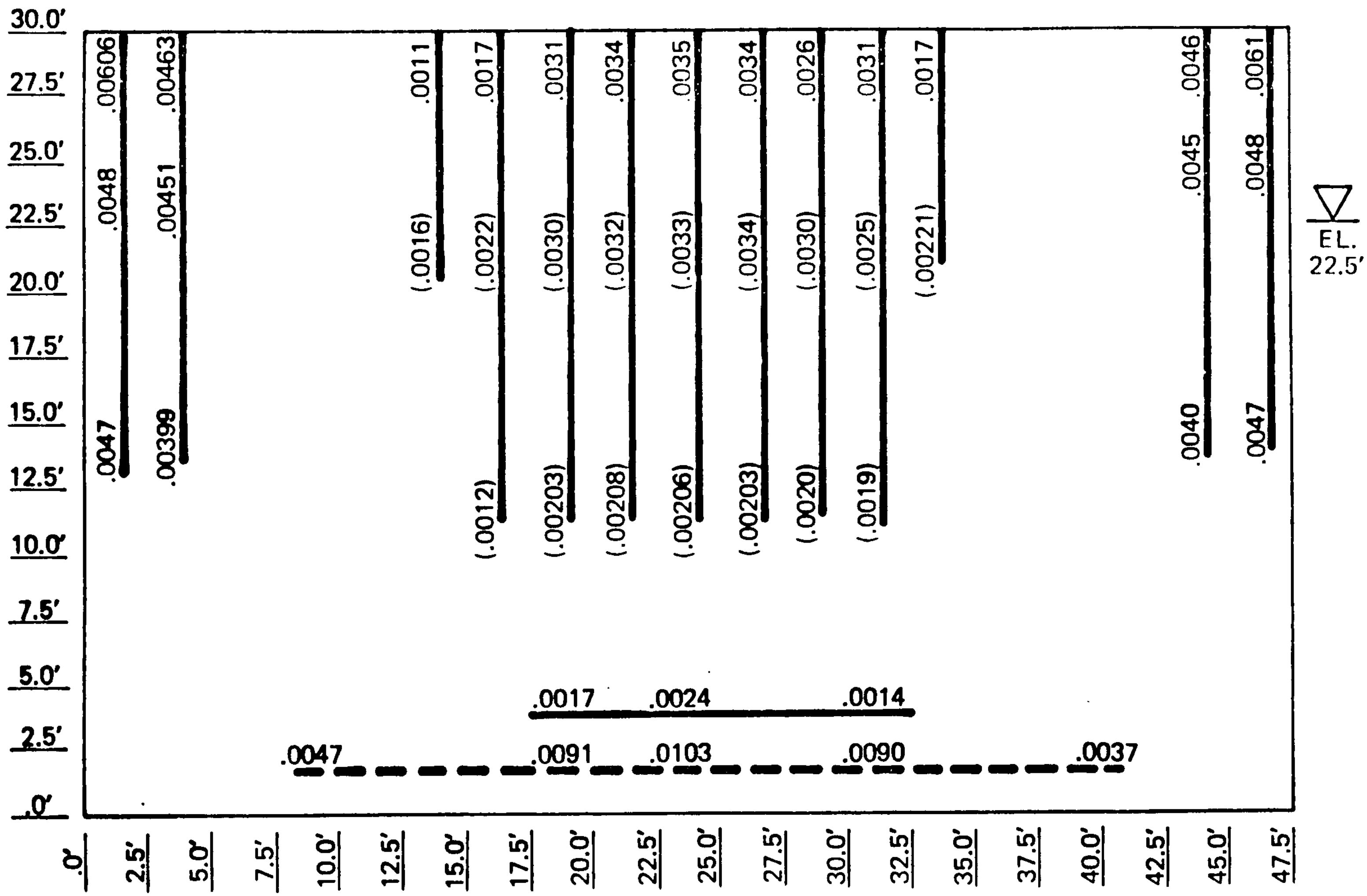

Figure 4.4 CONDENSATE STORAGE TANK AREA WALL (b)
DIAGRAM SHOWING CRACK APERTURES IN INCHES

Crack apertures shown relate to maximum aperture on inside wall face.
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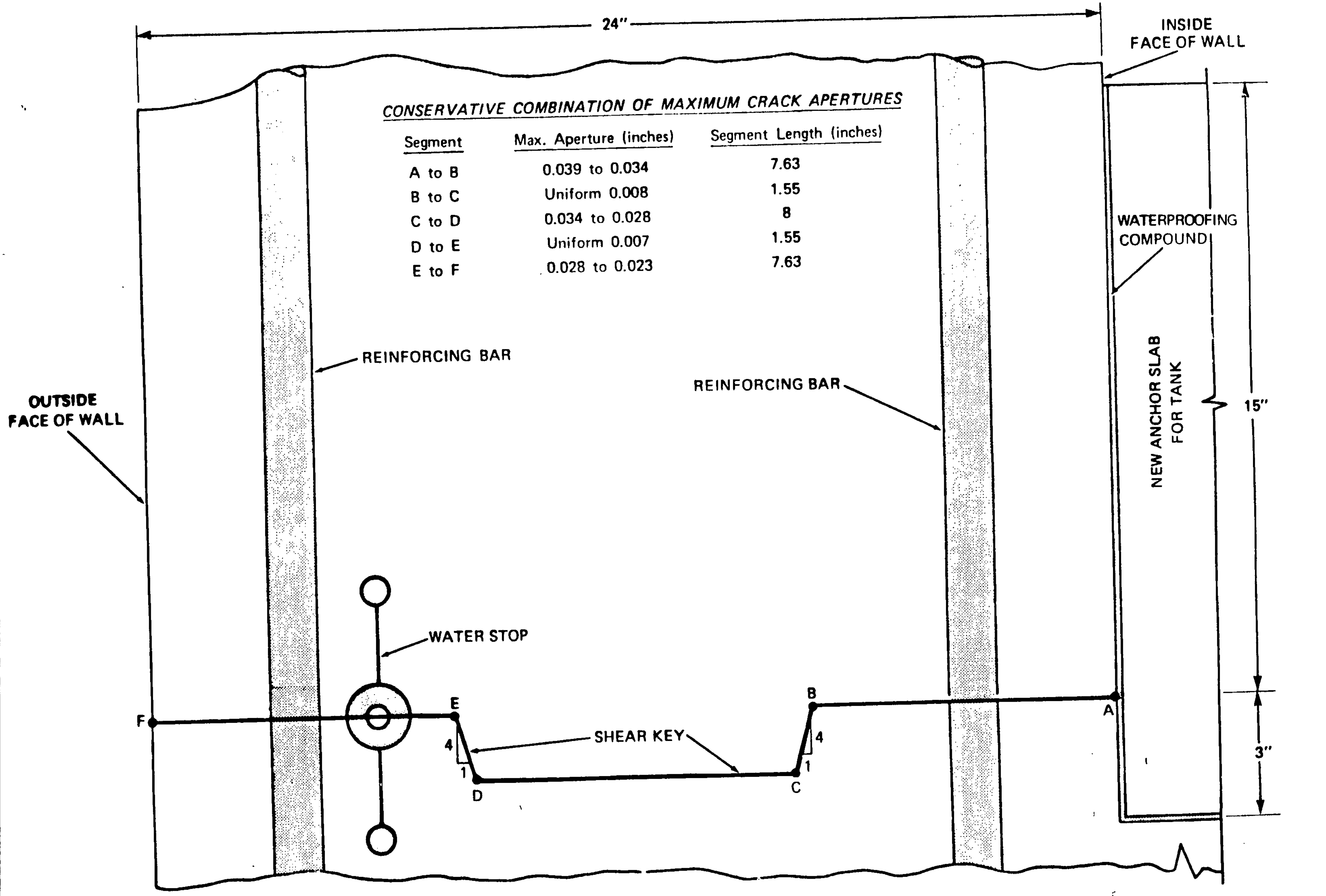
Bottom crack shown by a dashed line contains the water stop.



CONSERVATIVE COMBINATION OF MAXIMUM CRACK APERTURES

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D to E	Uniform 0.007	1.55
E to F	0.028 to 0.023	7.63

SCALE 1" = 3"

Figure 4.5 SECTION THROUGH BASE OF WALL

Appendix A

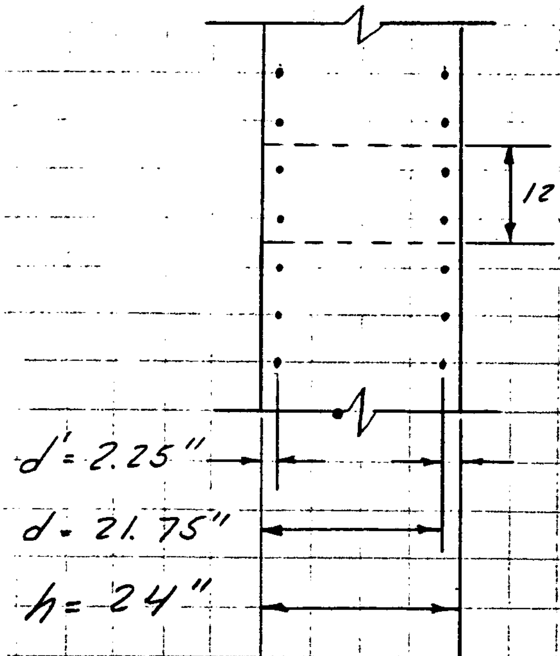
SAMPLE CALCULATION FOR VERIFICATION OF THE GOVERNING STRESSES / LOADS IN WALL ELEMENTS AND DETERMINATION OF CORRESPONDING WIDTHS OF CRACKS.

TWO REPRESENTATIVE LOCATIONS ARE CONSIDERED; ELEMENTS #9 & #134 AS IDENTIFIED ON SHEET 101.

ALL FORMULATIONS AND NOTATION ARE PER ACI DESIGN HANDBOOK (318-71) UNLESS OTHERWISE NOTED.

ELEMENT #9 (HORIZONTAL FLEXURAL & AXIAL LOAD)

1.0 TYPICAL SECTION



$f'_c = 4{,}000$ PSI

$f_y = 60{,}000$ PSI

#10 @ 6" EA. FACE

$A_s = A'_s = 2.54\ IN^2/FT$

$\therefore \rho = \rho' = \dfrac{2.54}{12(21.75)} = .00973$

SECTION A-A

LAO 0513 8-73

BECHTEL

CALCULATION SHEET

CALC. NO. C-259-05-05

SIGNATURE RMB for Eddy Alvarado DATE 9/18/80 CHECKED A.A.G DATE 9/22/1980

PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT STORAGE TANK BLDG - WALLS SHEET 89 OF 103 SHEETS

1.1 ULTIMATE MOMENT CAPACITY

PER FLEXURE 1.2 : (ACI DESIGN HANDBOOK (318-71))

$f_y = 60$ ksi & $\rho = .00973$

$$\rightarrow \begin{cases} a_u = 4.11 \\ \dfrac{c}{d} = .202 \\ K_u = 479 \\ .75\rho_b = .0213 \end{cases}$$

$$M_u = A_s(a_u d) = \underline{227 \text{ K-FT}}$$

$\rho = .00973 << .75\rho_b = .0213$

$\rho > \rho_{min} = .0033$

$\therefore$ M_u RESULTS BY YIELDING OF TENSION REINFORCING WITHOUT INITIAL CONCRETE CRUSHING.

PER FLEXURE 8 VERIFY THAT COMPRESSION REINFORCEMENT HAS INSIGNIFICANT CONTRIBUTION TO M_u, i.e: $M_{uc}/M_{uw} \simeq 1.0$:

$\rho = .00973$, $\dfrac{\rho'}{\rho} = 1.0$ & $\dfrac{d'}{d} = .103$

$$\rightarrow \frac{M_{uc}}{M_{uw}} = 1.0$$

$$\therefore M_u = M_{uc} = \left(\frac{M_{uc}}{M_{uw}}\right) M_{uw} = (1.0)(227) = 227 \text{ K-FT}$$

1.2 CRACKING MOMENT

$$M_{CR} = \frac{f_r I_G}{Y_t} \quad (\text{Eqn. 9-5 ACI 318-71})$$

$$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{4{,}000} = 474 \text{ PSI}$$

$$I_G = \frac{1}{12} \times 12 \times 24^3 + \underbrace{(8-1)}_{n}\underbrace{(2.54}_{A_s} + \underbrace{2.54)}_{A'_s}(9.75)^2$$

$$I_G = 13824 + 3380 = 17{,}204 \text{ in}^4$$

$$M_{CR} = \frac{f_r I_G}{Y_t} = \frac{474 \times 17{,}204}{12} = 679{,}57 \text{ lb-in}$$

$$M_{CR} = 57 \text{ 'K}$$

1.3 ELEMENT LOADS & CURVATURE

1.3.1 MAXIMUM SEISMIC LOADING COMBINATION

(FROM FINITE ELEMENT COMPUTER RUN NO. S1850 H.J DATED 6/23/80. VALUES OF M_x AND T_x ARE OBTAINED AS OUTPUT)

VALUES OBTAINED FROM COMPUTER RUN # S1850 H.J DATED 6/23/80:

$M_x = 144$ K-FT < 227 K-FT $= M_u$ ∴ ADEQUATE FLEXURAL CAPACITY

> 57 K-FT $= M_{CR}$ ∴ FLEXURAL CRACKING DEVELOPS

$T_x = 14.3$ Kips

$$f_{ST} = \frac{T_x}{A_{SE}} = \frac{14.3}{2(2.54)} = 2.8 \text{ ksi} \ll .9F_y = 54 \text{ ksi}$$

AND IT DOES NOT ENCROACH INTO AMPLE FLEXURAL MARGIN ABOVE

1.3.2 REVERSAL DUE TO SEISMIC INERTIAL LOAD OF WALLS:
(FROM COMPUTER RUN #S7120NJ DATED 6/11/80, LOAD 3.+.4 LOAD 4)

$M_x = 58.2$ K-FT $> M_{cr}$ ∴ OUTSIDE FACE IS CRACKED, ACCORDINGLY, THE COMPRESSIVE STRESS ACTING ON THE OUTSIDE FACE UNDER SUSTAINED HYDROSTATIC LOADING WILL BE CALCULATED TO EVALUATE MINIMUM CRACK WIDTH BETWEEN TWO FRACTURED SURFACES, SEE 1.3.3 FOR f_c CALCULATION.

1.3.3 SUSTAINED HYDROSTATIC LOADING:

VALUES OBTAINED FROM COMPUTER RUN #S3170IA DATED 7-24-80, LOAD 1 & RUN #SB5500O DATED 8-8-80

$M_x = 35.0$ K-FT $<< .4\, M_u = 91.0^{1K}$ SO THAT WSD METHODOLOGY FOR FLEXURAL STRESS EVALUATION AND LOCATION OF NEUTRAL AXIS IS APPLICABLE.

$T_x = 4.4$ K

ROTATIONAL RESPONSE AT NODES OF ELEMENT, FROM COMPUTER RUN OUTPUT.

$$\phi = \frac{(.000457-.000006)+(.000450-.000007)}{2} + \frac{(.000221-.000134)+(.000233-.000145)}{2}$$

$\phi = .000529$ RADIANS / OVER ELEMENT LENGTH

CALCULATION OF REINFORCING TENSILE STRESSES FOR SUBSEQUENT CALCULATION OF CRACK WIDTH PER ACI 224:

$$f_s = \frac{M_x}{A_s\, j\, d}$$

PER ACI WSD HANDBOOK SP-3:

TABLE 11: $m = n\rho + \rho'(2n-1) = 8(.00973) + .00973(2(8)-1)$

$m = .228$

$q = n\rho + \rho'(2n-1)\frac{d'}{d} = 8(.00973) + .00973(2(8)-1)\frac{2.25}{21.75}$

$q = .0929$

$\therefore k = .262$, $kd = 5.69''$

TABLE 12: $k = .262$ $\frac{1}{k}\left(\frac{d'}{d}\right) = .395$

$\frac{1}{k} \times \frac{(2n-1)A_s'}{bd} = .558$

$\therefore z = .358$

TABLE 13: $z = .358$, $k = .262$

$\therefore j = .906$, $jd = 19.71$

$f_{sb} = \frac{\overset{\text{sh. 91}}{35.0} \times 12}{(2.54)(19.71)} = 8.39 \text{ ksi} \ll .9F_y$ (FLEXURAL COMPONENT)

$f_{sT} = \frac{T_x}{(A_{sT})} = \frac{\overset{\text{sh. 91}}{4.40}}{2 \times 2.54} = .87 \text{ ksi} \ll .9F_y$ (AXIAL COMPONENT)

CALCULATION OF COMPRESSIVE STRESS IN CONCRETE TO VERIFY MINIMUM CRACK WIDTH BETWEEN FRACTURED CONCRETE SURFACES (WITHIN ZONE OF FLEXURAL COMPRESSION UNDER SUSTAINED LOADING).

$$f_{cb} = \frac{f_s}{n} \times \frac{k}{1-k} = \frac{8.39}{8} \times \frac{.262}{1-.262} = 372 \text{ psi}$$

$$f_{c_{net}} = f_{cb} - \left(\frac{1}{2} \text{ of NET SECTION}\right) = f_{cb} - \frac{1}{2}\,\frac{4.4}{5.69 \times 12 + (2 \times 8 - 1)2.54}$$

$f_{cNET} = 372 - 21 = 351 \text{ psi} > 250 \text{ psi}$, WHICH IS THE APPROX. LOWER BOUND REQUIRED TO JUSTIFY THE MINIMUM CRACK WIDTH OF 0.001"

VERIFICATION BY EQUILIBRIUM OF FORCES C & T:

$$\text{TOTAL } T = 8.38 \times 2.54 = 21.29 \text{ KIPS}$$

$$\text{TOTAL } C = .372 \frac{\times 5.69 \times 12}{2} + \left(\frac{5.69-2.25}{5.69}\right).372 \times 2.54(2 \times 8 - 1)$$

$$\text{TOTAL } C = 21.27 \text{ KIPS}$$

1.4 CRACK WIDTH CALCULATION

1.4.1 PER ACI COMMITTEE 224 & ACI 318-71 COMMENTARY SECTION 10.6.5:

$$w = k\,\beta\,f_s\,\sqrt{M_I}$$

$$M_I = \frac{d_{b1}\,S_2}{\rho_{I1}}$$

BECHTEL

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PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT STORAGE TANK BUILDING - WALLS SHEET 94 OF 103 SHEETS

WHERE $d_{b_1} = 1.25''$ $\beta = 1.30$

$S_2 = 12''$ $C_1 = 2.25 - \frac{1}{2}\left(\frac{10}{8}\right) = 1.625$

$$\rho_{C_1} = \frac{A_{s1}}{A_{C_1}} = \frac{2 \times 1.27}{12(2 \times 1.625 + 1.25)} = .047$$

$$M_I = \frac{1.25 \times 12}{.047} = 319 \text{ k-ft}$$

$$K = 2.8 \times 10^{-5}$$

$$\left.\begin{aligned} f_{sb} &= 8.39 \text{ ksi} \\ f_{sc} &= .87 \text{ ksi} \end{aligned}\right\} \text{SH. 92}$$

$$w(f_{sb}) = 2.8 \times 10^{-5} \times 1.3 \times 8.39 \sqrt{319} = .0054$$

$$w(f_{sc}) = 2.8 \times 10^{-5} \times 1.3 \times .87 \sqrt{319} = .00057''$$

$$w_f = .0054 + .00057 = .0060''$$

1.4.2 PER CURVATURE

$$\phi = .000529 \leftarrow \text{SH. 91}$$

$$w'_\phi = \phi(h - kd) = .000529(24 - 5.69) = .00969$$

CONTRIBUTION FROM AXIAL TENSION BASED ON f_{sb} AS ABOVE.

$$w_\phi = w'_\phi + w_{(f_{sc})} = .00969 + .00057$$

$$w_\phi = .0102'' > w_f = .0060'' \therefore w_\phi \text{ GOVERNS}$$

BECHTEL

CALCULATION SHEET

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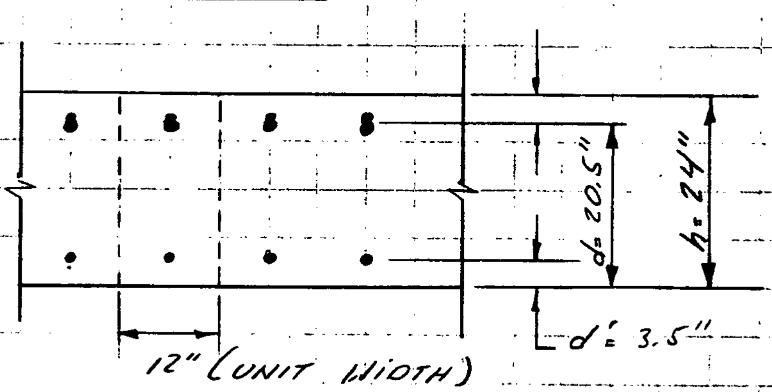
PROJECT SONGS 2 & 3 JOB NO. 10079-003

SUBJECT STORAGE TANK BUILDING - WALLS

ELEMENT #134 (VERTICAL FLEXURAL & NO AXIAL LOAD)

1.0 TYPICAL SECTION



SECTION B-B

COMPRESSION REINFORCEMENT : #10 @ 12"

$$A_s' = 1.27\ IN^2/FT\ ; \quad \rho' = \frac{1.27}{(12)(20.5)} = .00516$$

TENSION REINFORCEMENT : #10 @ 12"
#11 @ 12"

$$A_s = 2.83\ IN^2/FT\ ; \quad \rho = \frac{2.83}{(12)(20.5)} = .0115$$

1.1 ULTIMATE MOMENT CAPACITY

PER Flexure 1.2:

$f_y = 60$ ksi & $\rho = .0115$

$$\begin{cases} a_u = 4.05 \\ \frac{c}{d} = .240 \\ K_u = 559 \\ .75\,\rho_b = .0213 \end{cases}$$

$M_u \cdot A_s (a_u\, d) = \underline{235 \text{ K-ft}}$

$\rho << .75\,\rho_b = .0213$

$\rho > \rho_{min} = .0033$

$\therefore$ M_u RESULTS BY YIELDING OF TENSION REINFORCING WITHOUT INITIAL CONCRETE CRUSHING.

PER Flexure 8 verify THAT COMPRESSION REINFORCEMENT HAS INSIGNIFICANT CONTRIBUTION TO M_u:

$\rho = .0115$, $\frac{\rho'}{\rho} = \frac{.00516}{.0115} = .45$ & $\frac{d'}{d} = .171$

$\frac{M_{uc}}{M_{uw}} \approx 1.0$

$\therefore$ $M_u = M_{uc} = \left(\frac{M_{uc}}{M_{uw}}\right) M_{uw} = (1.0)(235) = 235$ K-FT

CALCULATION SHEET

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SIGNATURE Run for Eddy Awelaido DATE 9/18/80 CHECKED A.A.G DATE 9/22/1980

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SUBJECT STORAGE TANK BUILDING WALLS SHEET 97 OF 103 SHEETS

1.2 CRACKING MOMENT

$$\bar{Y} = \frac{\Sigma Ay}{\Sigma A} = \frac{\overbrace{19.81 \times 3.5}^{(n-1)A_s} + \overbrace{8.89 \times 20.5}^{(n-1)A_s'} + \overbrace{288 \times 12}^{b \times h}}{19.81 + 8.89 + 12 \times 24} = 11.70''$$

$$I_t = I_o + Ay^2 = \frac{12 \times 24^3}{12} + 288(13.75 - 11.7)^2 + 19.81(11.70 - 3.5)^2 + 8.89(20.5 - 11.70)^2$$

$I_6 = 17,055$ $y_t = 11.69$ $M_{cr} = \frac{474 \times 17,055}{11.69} = 691,529$ FT-lb

$M_{cr} = 58^{1k}$

1.3 ELEMENT LOADS & CURVATURE

1.3.1 MAXIMUM SEISMIC LOADING COMBINATIONS:

(FROM FINITE ELEMENT COMPUTER RUN NO. S1850 H.J DATED 6/23/1980)

$M_y = 104$ K-FT < 235 K-FT $= M_u$ ∴ ADEQUATE FLEXURAL CAPACITY.

1.3.2 REVERSAL DUE TO SEISMIC INERTIAL LOAD OF WALLS:

(COMPUTER RUN # S7120SJ)

$M_y = 80.7$ K-FT $> M_{cr}$

1.3.3 SUSTAINED HYDROSTATIC LOADING:

FROM COMPUTER RUN # S3170IA

$M_y = 18.3$ K-FT $<< .4 M_u = 94^{1k}$

$$\phi = \frac{(.00113 - .00082) + (.00113 - .00082)}{2}$$

$\phi = .000310$ RADIANS / OVER ELEMENT LENGTH

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SUBJECT STORAGE TANK BUILDING - WALLS

CALCULATION OF REINFORCING STRESSES FOR SUBSEQUENT CALCULATION OF CRACK WIDTH PER ACI 224:

$$f_s = \frac{M_y}{A_s j d}$$

PER ACI WSD HANDBOOK SP-3:

TABLE 11: $m = 8(.0115) + .00516(2(8)-1) = .169$

$q = 8(.0115) + .00516(2(8)-1)\left(\frac{3.5}{20.5}\right) = .105$

$\therefore\ k = .319 \qquad kd = 6.55''$

TABLE 12: $k = .319 \qquad \frac{1}{k}\left(\frac{d'}{d}\right) = .534$

$$\frac{1}{k} \times \frac{(2n-1)A_s'}{bd} = .242$$

$\therefore\ z = .370$

TABLE 13: $z = .370, \quad k = .319$

$\therefore\ j = .882 \qquad jd = 18.1$

$$f_{sb} = \frac{18.3 \times 12}{(2.83)(.882)(20.5)} = 4.30\ ksi << .9F_y$$

(FLEXURAL COMPONENT)

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FOREGOING IS CONSERVATIVELY HIGH VALUE FOR TENSILE STRESS NEGLECTING AXIAL COMPRESSION DUE TO WEIGHT OF WALL.

FOR COMPRESSIVE STRESS AT FRACTURE CONCRETE SURFACES (WITHIN ZONE OF FLEXURAL COMPRESSION UNDER SUSTAINED LOADING CONDITION) CONSIDER CONTRIBUTION DUE TO WEIGHT OF WALL.

WEIGHT of WALL $N = 26.25 \times 1.0 \times 2.0 \times 150 = 7.88$ kips

$b = 12''$, $d = 20.5''$, $d' = 3.5''$, $d'' = 8.5''$

$m = 8$, $A_s = 2.83''^{\square}$, $A_s' = 1.27''^{\square}$, $\dfrac{d'}{d} = .171$

SH. 97

$M_y = 18.3$ FT-K

$$e = \frac{12M}{N} + d'' = \frac{12 \times 18.3}{7.88} + 8.5 = 36.37 \quad \therefore \frac{e}{d} = 1.77$$

FROM TABLE 10a, for $\dfrac{e}{d} = 1.77$ $j = .88$ (ESTIMATED) :

(ACI WSD HANDBOOK SP-3)

$i = 1.99$

$$m = \frac{nA_s i}{bd} + \frac{(2n-1)A_s'}{bd} = \frac{(8)(2.83)(1.99)}{(12)(20.5)} + \frac{(2 \times 8 - 1)1.27}{(12)(20.5)}$$

$$m = .183 + .077 = .26$$

$$q = \frac{nAi}{bd} + \frac{2n-1}{bd} \times \frac{d'}{d}$$

$$q = .183 + .077 \times (.171)$$

$$q = .196$$

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FROM TABLE 11, for $m = .26$ and $g = .196$

$$\therefore k = .418$$

FOR ENTERING TABLE 12, COMPUTE

$$\frac{1}{k} \times \frac{(2n-1)A'_s}{bd} = \frac{1}{.418} \times .077 = .184$$

$$\frac{1}{k} \times \frac{d'}{d} = \frac{1}{.418} \times .171 = .409$$

$$z = .35$$

FROM TABLE 13, for $z = .35$ AND $k = .418$

$$j = .85$$

FROM TABLE 10A: for $\frac{e}{d} = 1.77$ AND $j = .85$

$$i = 1.92$$

$$f_s = \frac{N}{j A_s i} \times \frac{e}{d}$$

$$f_s = \frac{7.88}{(.85)(2.83)(1.92)} \times 1.77$$

$f_s = 3.02$ ksi WHICH AS ANTICIPATED IS LESS THAN $f_{sb} = 4.3$ (PER SH. 98) USED

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CALCULATION of COMPRESSIVE STRESS IN CONCRETE:

$$f_c = \frac{f_s}{n} \times \frac{k}{1-k} = \frac{3020}{8} \times \frac{.418}{1-.418}$$

$$f_c = 271 \text{ psi} \gtrsim 250 \text{ psi}$$

1.4 CRACK WIDTH CALCULATION

1.4.1 PER ACI 224:

$$w = K\, \beta\, f_s \sqrt[3]{M_I}$$

WHERE $d_{b1} = \frac{1\frac{1}{4} + 1\frac{3}{8}}{2} = 1.31''$

$$S_2 = 6''$$

$$\beta = 1.3$$

$$\rho_{T1} = \frac{A_{S1}}{A_{T1}} = \frac{1.27 + 1.56}{12(2 \times 2.75 + 1.31)} = .0345$$

$$M_I = \frac{1.31 \times 6}{.0345} = 228$$

$$K = 2.8 \times 10^{-5}$$

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sh. 98

$f_{sb} = 4.30$ ksi

$w_{f_{sb}} = 2.8 \times 10^{-5} \times 1.3 \times 4.3 \times \sqrt[3]{228} = .0024''$

1.4.2 PER CURVATURE:

sh. 97

$\phi = .00031$

$w_\phi = \phi(h - kd) = .00031(24 - 6.55)$ (NO AXIAL TENSION, THEREFORE NO ADDITIONAL MOMENT)

$w_\phi = .0054'' > w_{f_{sb}} = .0024''$ $\therefore$ w_ϕ GOVERNS

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SUBJECT STORAGE TANK BLDG. WALLS

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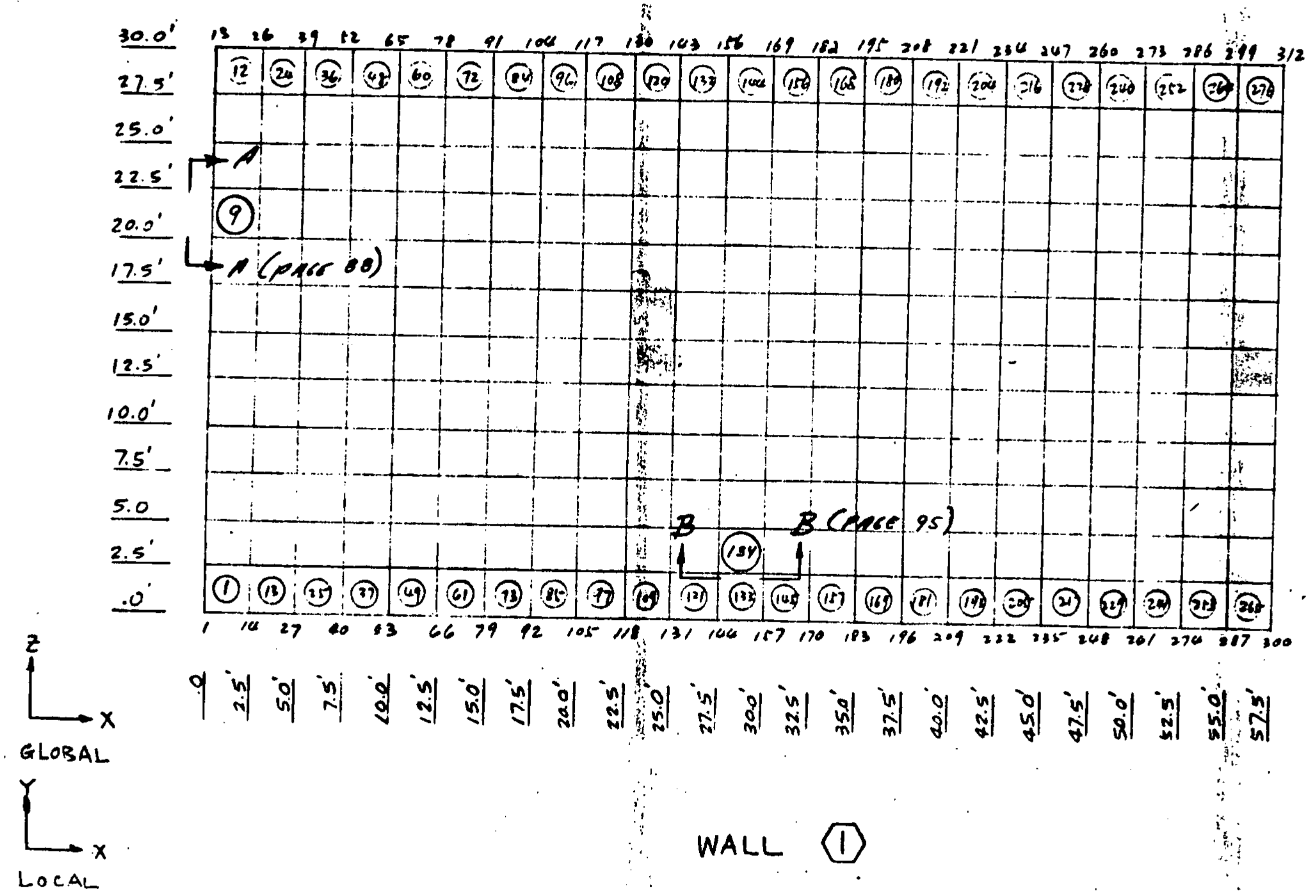

WALL 1