SEISMIC FAILURE AND REPAIR OF AN ELEVATED WATER TANK

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SUMMARY

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B. Soil Conditions
C. Seismic Behavior
D. Proposed Repairs
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A. Description of Damages

Bueras Water Tank, a structure 41.6 m. in height with 4000 m^3 capacity, located at Valdivia, Chile, suffered spectacular damages during the earthquakes of May,1960.

The tank was built during 1958-1959, at an estimated actual cost of US$300.000. Between August 31 and September 30, 1959, it was filled with water for hydraulic tests, and it has not been used since. The seismic shocks of May 21 and 22, 1960, found it empty.

The elevated tank's structural failures are due to two independent causes: excessive soil settlement and extensive seismic damage. (1, 2, 3, 4, 5).

Settlements were measured on eight points distributed around the base, during a seven-year period, from October 31st, 1958, to May 5th, 1965. Figure 1 shows the location of control points and the time settlement curves. It can be seen that the tank sank to a maximum of 17.9 cm. in 1965, but that differential settlement remained constant at 2.35 cm., during the last five years of control. It is also noted that approximately 1/3 of the settlement took place during the 34 days the tank was filled with water in 1959. The 1960 earthquakes did not apparently influence the rate of movement, except for minor irregularities probably due to variations at the reference points. At the time of the last control, the phenomenon was still in progress, at a rate of 10 mm. per year.

In May 1960 the tank suffered the effects of the series of earthquakes that struck the Southern part of Chile; these earthquakes are among the greatest in seismic history and have been judged to be more than the sum of all shocks experienced in California since 1900 (6).

The two main shocks were felt at Concepción on May 21st and at Valdivia on May 22nd. The latter had a Richter magnitude of 8.5 and Modified Mercalli Intensity of 10 in Valdivia; it originated at about 200 Km. north of the town, and the faulting is believed to have progressed along the coast, to the south, over a distance of 800 Km. The town of Valdivia, located on the coast, suffered a general

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tectonic subsidence of 1.5 m. and was struck by tsunamis (6, 7).

Damages suffered by the tank's structure are shown in Figures 2 and 3 and are described below:

a.- Small cracks inside the upper tank.

b.- A large number of vertical, horizontal and diagonal big cracks in the tower mantle and fins, that go through the wall thickness and leave the reinforcing bare.

c.- A large number of horizontal cracks in the radial fins, at approximately 5 m. from the ground, coinciding with a construction joint and a zone of heavy splicing of vertical reinforcing.

d.- Vertical cracks separating the fins from the mantle.

e.- Deep diagonal cracks at the lower part of the fins and separation between them and the foundations, with severed steel bars and appreciable offset; and

Radial cracks on the footing's upper face.

B. Soil Conditions

An extensive program of soil explorations and testing was carried out between 1961 and 1965. The first studies, done by the University of Chile, included one deep Standard Penetration Test and a series of Classification Tests and Consolidation Curves; four additional Penetration Tests, as well as Laboratory Checks were made by the writer's office in 1965, under the direction of Sergio Della Maggiora, Soil Mechanics Consultant, who is responsible for the analysis done in this paragraph.

Figure 4 shows a typical soil profile under the tank; three different layers are clearly discernible:

a.- Elevation 0 to -10 m.:
Highly compressible yellow silts, classification MH and ML with some organic matter. Low to medium consistency, high void ratio and sand content below 10%. Interposed in this stratum, immediately below the footing, there is a hard layer of cemented sandy silt of 0.8 m. average thickness, classification SM, of very good bearing capacity, locally known as "Cancagua".

b.- Elevation -10 to -15 m.:
Hard grey sandy silts of low compressibility, classification ML, sand contents from 12% to 45%; and

c.- Elevation -15 to -22m.:
Grey fine dense sands and silty sands, SP and SM.

An analysis of bearing capacity and settlements was made of these soils. As a first approximation, bearing capacity was computed by means of Terzaghi's classic formulas using static seismic forces (8); water masses were reduced in accordance with the theory developed by A. Arias and S. Moran (9); the effect of load excentricity due to differential settlements was also considered.

Two different seismic factors were assumed: a normal coefficient of 0.2 g. following the requirements of most seismic codes (10,11,12) and an extreme factor of 0.6 g. believed to be close to the maximum acceleration reached at top levels. (See paragraphs C and D below). The computed safety factors against rupture are summarized below; normal factors recommended by Terzaghi are also shown (8).

Condition		Safety Factors Computed	Safety Factors Normal
Gravity loading,	empty tank	4.0	3
	full tank	2.5	3
0.2 Earthquake,	empty tank	4.0	2
	full tank	2.3	2
0.6 Earthquake,	empty tank	2.0	
	full tank	0.8	

The results are consistent with the fact that no rupture of the soil has been observed; they also indicate that the full tank does not have sufficient safety for gravity loading in a full condition.

The safety factor of 0.8 under a 0.6 earthquake does not necessarily indicate a danger of failure; cohesive soils are known to have considerable strength for dynamic loads provided there is no change in their basic structures.

Even though the static method of seismic analysis is of limited value, it is considered sufficient for the purposes of this investigation.

Soil settlements were analyzed on the basis of 17 Consolidation Tests. Most settlements are due to the upper layer of compressible silt. Maximum computed soil deformation with the tank empty is 23.5 cm., somewhat higher than the value of 17.9 cm. measured in 1965 (Fig. 1). It was also determined that settlements for the full tank may reach 36.5 cm. maximum and 7.0 cm. differential; these figures may be subject to considerable variation because soil characteristics are far from uniform.

To summarize, the full tank under normal conditions is not adequately safe and is subject to excessive settlements; it may also cause rupture of the soil during strong earthquakes.

The tank is a tall, heavy and expensive structure (41.6 m. 11200 M.T. with water, US$300.000), that, because of its critical importance during emergencies, should have higher than usual safety factors. Therefore, it was considered necessary to reinforce its foundations before putting it into service.

C. Seismic Behavior

Because no strong motions recordings of the 1960 earthquakes are available, the mechanism of the observed failure was investigated with the two following spectra:

- El Centro, California, May 18, 1940, N.S. Component; and
- Standard Spectrum suggested by Esteva and Rosenblueth at the first Chilean Conference on Earthquake Engineering in 1963 (13).

As reference, a comparison between the main shocks of 1960 and El Centro is given below: (1, 3, 6).

	El Centro	Valdivia	Concepción
Richter, Magnitude	7.1	8.5	7.5
Intensity, MM	7.5	10	9
Estimated Ground Acceleration	0.33 g.	0.35 g.	0.30 g.
Distance to Epicenter, Km.	25	100	80
Depth of Focus, Km.	50	24	50

The main periods of vibration of the undamaged tank, computed by the Raleigh Method for four different conditions, are as follows:

No soil rotation,	empty tank	0.20 sec.
	full tank	0.32 sec.
1/200 elastic rotation,	empty tank	0.80 sec.
	full tank	0.90 sec.

The soil rotation of 1/200 is based on the observations of C.M. Duke, who measured a 2 cm. gap between the earth and the fins immediately after the earthquakes (3, 14).

The periods of two similar elevated tanks, built in Santiago, are: (15)

Tank	Period	Damping
Presidente Ríos, full	0.385 sec.	1.4% critical
Cerro Navia, full	0.225 sec.	1.54% critical

These tanks are built on compact gravel, and soil rotations during tests can be assumed to be negligible. The computed period of 0.32 sec. falls within the range of the experimental values.

The period of the damaged structure was then computed, assuming the following conditions, that are somewhat justified by the observed cracks (Fig. 2):

a.- Supporting tower consisting of 10 fins and 10 circular sectors, with full vertical separation between each other; and

b.- Supporting tower consisting of 10 independent circular sectors; the fins are considered as non active because the large cracks between elevations 0 and 5 m. prevent their structural action.

The theoretical periods are compared below with the results of field tests made by W.K. Cloud in November 1960. (4)

Period of Empty Damaged Tank	No Soil Rotation	1/200 Soil Rotation
Assumption a, 10 fins and 10 sectors	0.43 sec.	1.70 sec.
Assumption b, 10 sectors	0.67 sec.	2.70 sec.
Measured by W.K. Cloud	1.30	

Because soil rigidity during the small oscillation tests is higher than at the time of the earthquakes, and because actual damages are not as severe as the extreme condition assumed, it is logical to expect a measured period smaller than the maximum computed values.

Seismic accelerations for the tank were then determined, applying the probable periods to the selected spectra, using a damping coefficient of 1.4% of critical, based on the tests run in Santiago (15). For these purposes the structure was treated as a one degree of freedom inverted pendulum, with 50% of the weight of the tower concentrated at the upper level.

The following maximum accelerations were thus obtained:

Condition	Soil Rotation	Period	El Centro	Rosenblueth
Sound tank, Empty	1/200	0.80 sec.	0.70 g.	0.60 g.
Sound tank, Full	1/200	0.90 sec.	0.60 g.	0.60 g.
Damaged tank, Empty		1.30 sec.	0.25 g.	0.30 g.

Computations to determine the ultimate strength of the structure and its main components were then made, on the basis of standard theory and the strength of materials controlled during construction. The following main conclusions were reached (see Fig. 2):

a.- The foundations did not have adequate capacity for negative bending moments due to vertical loads.

b.- The weakest sections of the tank were the vertical joints between the fins and the mantle. Their ultimate bending moment capacity is reached with horizontal acceleration between 0.03 g. and 0.05 g.; and

c.- Should the structure maintain its integrity, ultimate capacity of the full tower section would be reached with seismic accelerations of o.44 g. for the empty tank and 0.15 g. for the full tank.

Taking into consideration the probable seismic accelerations and the computed strength, the sequence of failures may be hypothetized to have been the following:

a.- Foundation cracks, at the upper face of the footings occurred before the earthquake, probably during the test filling in September 1959.

b.- Vertical cracks appeared between the mantle and the fins, during the first few seconds of the earthquake.

c.- The appearance of these cracks increased the period, thereby decreasing maximum accelerations.

d.- After the mantle and fins were separated, the structure behaved in an altogether different manner from that assumed by the designer, and the other damage took place. At this stage, the period must have been higher than the 1.30 sec., measured by W.K. Cloud for small oscillations, and extensive yield of reinforcing as well as internal friction at the cracks must have occurred. The combined effect of these two factors probably prevented maximum acceleration from rising above 0.25 g. or 0.30 g., keeping it below the dangerous value of 0.45 g., thus saving the structure from total failure.

Likewise it can be stated that, had the tank been full, it would probably have collapsed.

D. Proposed Repairs

The required repairs fall into two categories:

a.- Solution of the foundation problems, mainly excessive soil settlements and inadequate structural capacity for normal loadings (full tank). Foundation weakness is independent of seismic conditions; and

b.- Repairs of the supporting tower.
Because of the time elapsed since the earthquake, (six years when this analysis was made) corrosion of the reinforcing steel in the open cracks is widespread, and direct repairs by means of pressure grouting or other conventional methods is not practicable.

The proposed strengthening, therefore, consists on the construction of a new system of supports that involves piling, heavy construction of tall structures, transfer of large loads and other slow and difficult operations. Several basic schemes were considered and two main ones were selected for preliminary design. Cost estimates, as well as other considerations, lead to the choice of one solution, for which complete drawings and specifications were prepared. At the time of this writing, bids for the reparation are scheduled for 1969 and the final decision on the tank's future will depend on their results. Nevertheless, on the basis of the official estimates, repair seems to be preferable to demolition and construction of a new tank.

The following design conditions were chosen on the basis of the analysis made in paragraph B and C of this paper:

a.- Normal loading
Dead load and water weight; design with basic units stresses.

b.- Accidental loading
Wind (does not control) or 0.20 g. earthquake. Allow 33 1/3% increase of units stresses for reinforced concrete and 50% for soils and piling. The seismic coefficient was chosen after compairing the following codes: (10, 11, 12)

Code	Seismic Coefficient	% Increase of Unit Stresses	
UBC	0.20	all materials	33 1/3
France up to 10 m.	0.11	steel	50
Max. at top	0.19	concrete	300
Japan up to 16 m.	0.40	all materials	50
Max. at top	0.54		
Mexico, Federal District	0.196	steel	50
		concrete	100
USSR base	0.10	steel	40
top	0.16	concrete	20

c.- Ultimate loading
El Centro and Esteva-Rosenblueth Spectra with 1.4% of critical damping and safety factor higher than 1.0 using ultimate strength design as per ACI code.

Alternate solution 1 is illustrated in Figure 5; in it, the tower is replaced by a cylindrical shell and piles are driven through the footings. The sequence of construction is the following:

a.- A new cylindrical reinforced concrete shell is built outside and around the old one, that is used as a form. Fins must be perforated to permit construction.

b.- A system of upper radial beams and a central reinforced concrete ring is built to support the tank's bottom and dome.

c.- Foundations are reinforced driving 154 cast in place 105 M.T. piles to elevation -10.0 m. and increasing the height of the footing.

d.- Old mantle and fins are demolished.

In solution 2, shown in Figure 6, a new foundation is built around the existing one, connected to the tank with an inverted truncated cone tower. Proposed construction sequence is;

a.- Build new foundations supported on 165 piles, 105 M.T. each. As an alternate, direct footings to elevation -10.0 m. were considered and discarded because of higher cost and the danger of undermining the tank.

b.- Build the truncated cone tower shell; two alternates were considered, solid wall and rigid frame, and the first one selected because of simplicity and economy.

c.- Build the system of brackets and circular beams to support the tank's bottom. This system is offset in relation to the fins to avoid interferences. Brackets proved to be less expensive than the radial beams of solution 1.

Shear keys and pressure grouting are specified for effective transfer of loads.

d.- The existing shell and fin are demolished.

The following cost estimates were prepared:

- Alternate 1	US$ 205,000
- Alternate 2	US$ 183,000
- Cost of demolition of existing structure and construction of a new tank:	US$ 312,000

Alternate 2 was chosen because, aside from the low cost, it allows for safer and cleaner construction with less interferences with the existing structure. Pile drivings through the footings, as proposed in solution 1, was considered difficult and dangerous.

A number of construction precautions and controls are required. The most important refer to the critical stages of load transfer to the new tower, prior to demolishing the old one. The following method was specified·

a.- The new tower and foundations are designed with sufficient capacity to support simultaneously the weight of the tank, 50% of the water, and the old shell, fins and footings hanging from it. This "erection" condition is not critical when compared to normal and earthquake loadings.

b.- After connecting the new tower to the tank and before removing the scaffolds, a number of control points are marked and levelled in the existing as well as the new foundations.

c.- When concrete is set, scaffolds are removed. At this point a small part of the weight may be assumed to be transferred to the new supports.

d.- Water is pumped inside the tank in 4 stages, 1 m. in depth each, reaching 50% of the capacity (2000 T).

e.- Water load is maintained a minimum of 30 days.

f.- The old shell and fins are removed.

During the entire process, following each step and before the next one, a careful inspection of the structure and control of the bench marks are specified.

The procedure is based on the known soil characteristics and on the experience gained from the test filling of September 1959. (See paragraph A and Fig. 1). It is, therefore, expected that the gradual loading will effectively transfer the weight to the new tower, whose foundations are considerably more rigid. As a precaution, the design condition described in point 1 precludes overloading of the newssupports.

When there is certainty that load transfer has been achieved, the old mantle and fins should be eliminated for safety purposes and the tank can be filled to capacity.

E. Conclusion

Major earthquakes and destruction of expensive and critical constructions such as elevated water tanks are unfortunate events. Nevertheless, modern engineering offers the necessary resources to save structures as severely damaged as the Bueras Tank, at reasonable cost. It is believed, furthermore, that most structures that are condemned in the crisis that usually follows major earthquakes could be economically saved. It is the author's hope, therefore, that this paper will be of use to engineers facing similar problems in the future.

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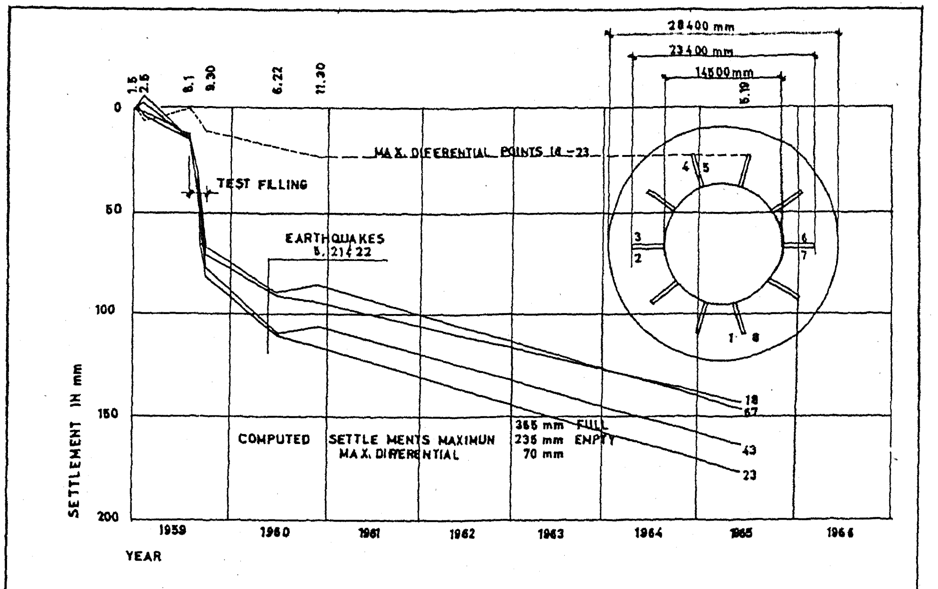

FIG 1.- BUERAS TANK - SETTLEMENT OF FOUNDATIONS

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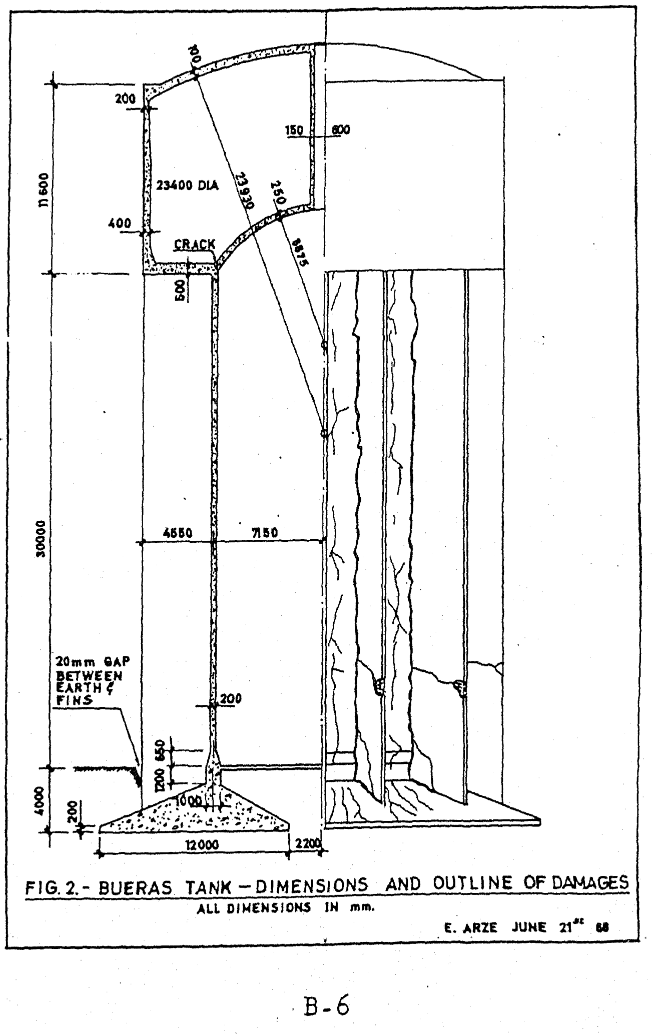

FIG. 2.- BUERAS TANK - DIMENSIONS AND OUTLINE OF DAMAGES

ALL DIMENSIONS IN mm.

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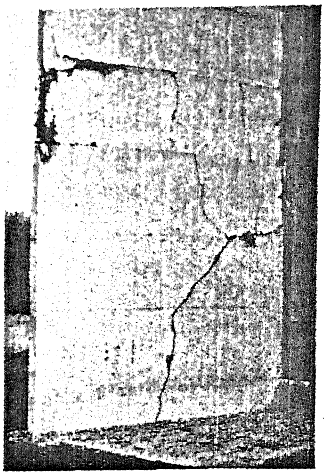

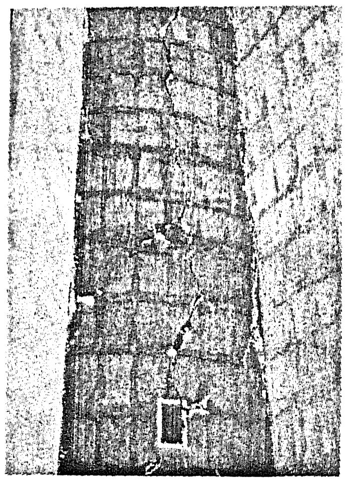

FIGURE 3 - BUERAS' TANK - PICTURES OF DAMAGED TANK

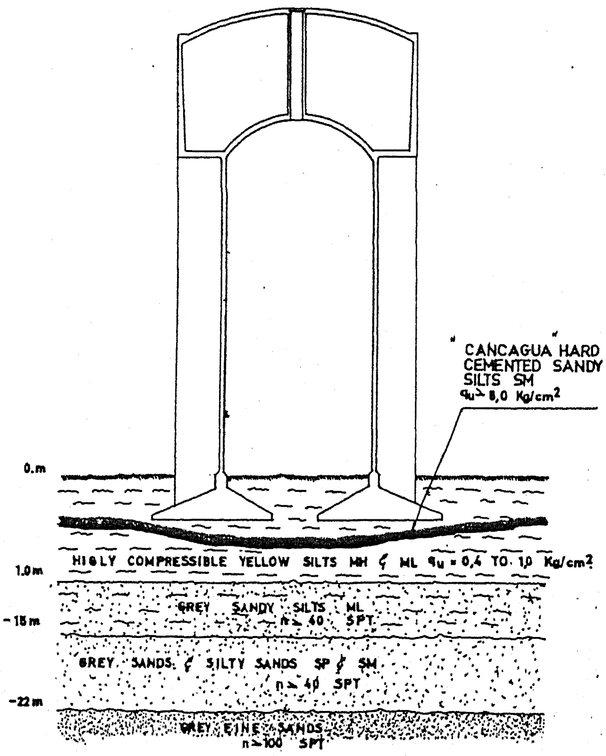

FIG. 4.- BUERAS TANK-SOILS PROFILE

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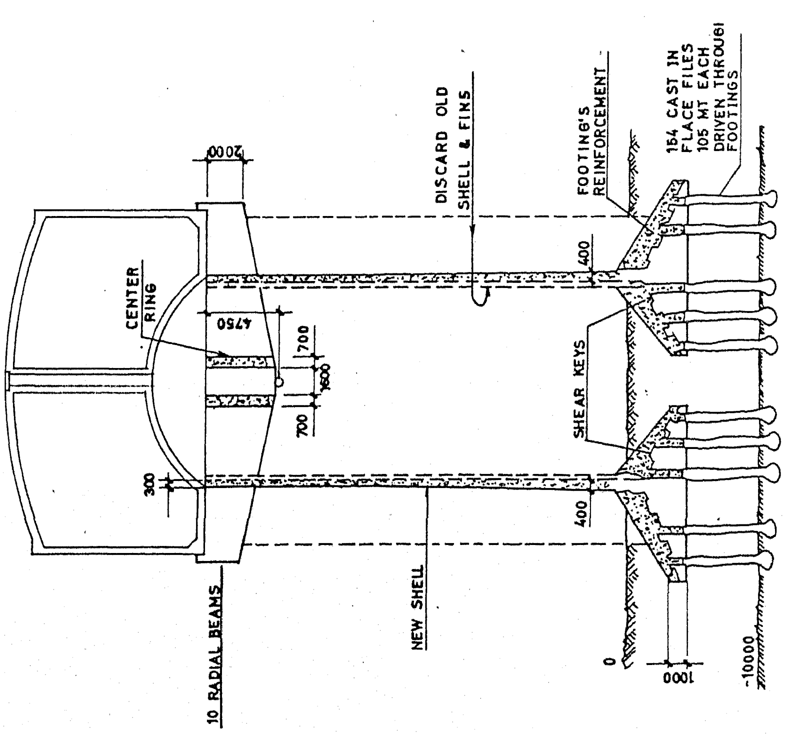

FIG. 5.- BUERAS TANK REPAIR - ALTERNATE SOLUTION Nº1
ALL DIMENSIONS IN mm

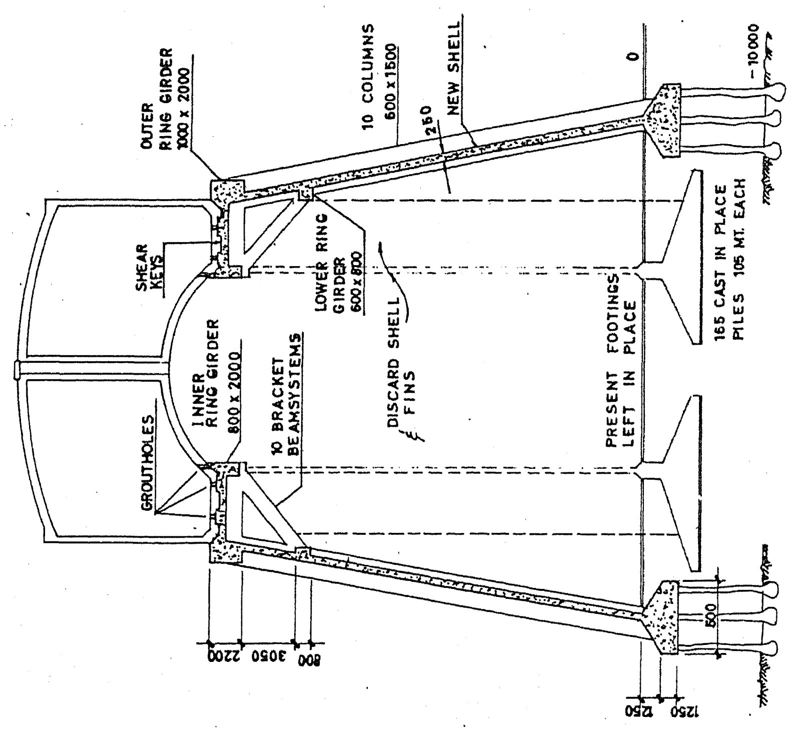

FIG. 6.- BUERAS TANK REPAIR - SOLUTION N° 2
ALL DIMENSIONS IN mm