THE ORNL RADIOACTIVE CARRIER DROP TEST FACILITY: ITS DESIGN, CONSTRUCTION, AND OPERATING CAPABILITY*

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ABSTRACT

A large drop test facility has been constructed at the Tower Shielding Facility (TSF) at the Oak Ridge National Laboratory. A 670- metric ton impact pad was constructed of reinforced concrete and armor plate and located between two 96-m-tall towers of the TSF. The towers, with heavy-duty hoisting equipment attached at their tops,are 39 m apart and are capable of lifting and dropping casks from heights of 60 m. Casks weighing 100 metric tons can be tested at 9 m, whereas lesser weight casks can be handled at greater heights. This paper describes the design, construction, and operating capability of this facility, unique in the free world.

INTRODUCTION

In October 1975, the Oak Ridge National Laboratory approved the design, construction, and installation of a permanent drop pad at the Tower Shielding Facility (TSF). This installation was to replace the small, temporary drop pad at the TSF.

The structure would be a reinforced-concrete, truncated Aztec pyramid, 8 m by 15 m at the base, 4 m by 8 m at the top, and with a total height of 3 m. The impact surface, a 64-metric ton (MT) structure, 5-1/2 m long, 2-1/2 m wide, and 0.63 m high, would be constructed of laminated steel (four layers, each 15.88 cm thick). The TSF site was selected because of the existing availability of hoisting equipment that was capable of lifting large loads to great heights, its remote location atop a small mountain in a controlled access area, and its underground control room for remote operation. The TSF consists of four towers, each of which is 96 m high, set in a rectangular array 30 m by 60 m (see Fig. 1). Each of the towers is guyed with two pairs of 5-cm-diam cables. The hoisting system is connected to the top of each tower. The impact surface would be located midway between towers III and IV, which are 60 m apart.

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TOWER SHIELDING FACILITY LIFTING CAPABILITY

The determination of the capabilities for lifting casks using the TSF is based on the assessment of the load-carrying capabilities of each of the TSF components: inclined guy cables (stay-type cables between the tip of each tower and the ground), horizontal tie cables (connecting cables at the top of the towers), structural members of the tower legs, dynamic loading associated with the cable snap-back occurring at the time of cask release, soil-bearing capacity at the footings of each of the tower legs, and loads in the supporting sheaves.

In addition to the above considerations, the tower capabilities are constrained on the basis of the system kinematics. The main contributing factors associated with these considerations are sag of the horizontal tie cables (the displacement in the vertical direction of the midpoint of the horizontal cable as measured from the unloaded tower configuration), tip-tower displacement, and loss of preload in the horizontal cable support system.

A functional relationship involving cask weight, load in the horizontal tie cables, and drop height was developed as shown by Eq. (1).

$$W = [(90.221 - h)/29.108][21.567 - 0.068T + 159.572/T^2] + 4.103 + [1.834 - 0.0368h + 0.000204h^2]^{1/2} \qquad (1)$$

where

W = cask weight (metric tons),

h = drop height (meters), and

T = tensions in horizontal tie cables (metric tons).

Equation (1) is the functional relationship that was used in a parameter study to investigate the tower lifting capabilities. The drop height vs cask weight relationship was determined on the basis of Eq. (1) by first considering the events that produce overloading on any of the system components. Only the results of the full parametric study conducted are presented here. These results are based on the following limitations associated with the structure, constrained by a set of safety factors which were imposed in the analysis.

(1.) The maximum load of each of the tower legs, in the vicinity of the base, was limited to 346,545 kg, which was the upper limit established in the original design of the facility.

(2.) The lifting cable system is limited by a safety factor of 6, based on the ratio of maximum line pull to nominal breaking load of the cable. Because the steel guy cables were considered to have the same safety factor, their maximum working load was equal to 28,123 kg (168,736/6).

(3.) The maximum bearing load of the soil at the base of the smallest of the two footings has been calculated to be equal to 274,332 kg, which was based on the original design allowable soil-bearing value of 20,506 kg/m^2. Subsequent field work has yielded soil bearing values which are almost twice those used in the initial design.

(4.) The stresses in the tower leg structural components due to the dynamics associated with the snap-back of the cables that occurs immediately after release of the cask is limited to 5062 MT/m^2.

(5.) The loss of preload on the horizontal tie cables is not a primary controlling factor, since loss of preload occurs only after the inclined guy cables have been overloaded with respect to the normal safety factor of 6.

Figure 2 is a graph of the parametric study using Eq. (1), which for this purpose was based on a maximum load in the inclined cables of 28,123 kg. The initial section of the curve, up to h = 15.24 m, is limited by the static soil-bearing pressure of 20,506 kg/m^2. The curve from points A to B represents cables associated with a safety factor of 6. Point C is the point at which the maximum impulse is imparted to the impact pad.

ANALYSIS AND DESIGN OF THE CASK IMPACT PAD

Based on the analysis of the TSF previously described, the design loading of the cask impact pad was a 78.4-MT cask dropped from a height of 30.78 m. This combination produces the largest impulse (176,130 kg-sec). The design approach was to provide the flat horizontal impact surface by embedding steel armor plates in a concrete mass. Since the site was fixed, this mass had to rest on approximately 12 to 15 m of overburden. Static-soil tests were made which indicated that the ultimate bearing capacity ranged from 102 to 131 MT/m^2. Based on the average value of the tests, 122 MT/m^2 was chosen as the design load on the rigid-pad base. Assuming this load, the pad was designed using the American Concrete Institute's working stress method for conventional isolated rectangular footings. This conservative approach was considered necessary to ensure that the pad with embedded plates was rigid. This rigid mass resting on the soil was then idealized as a single-degree-of-freedom spring-mass system under impulse loading. A generalized linear impulse theory for undamped systems[1] was used to determine the peak spring deflection. The resulting peak dynamic spring force was compared with the stated design load on the pad base to verify the design.

Refraction seismograph techniques were used to provide an estimate of the dynamic shear modulus of the soil. This technique measures the compressional (P-wave) wave velocity. Relationships for p-wave velocities, shear-wave velocities, and Poisson's ratio were used[2] to arrive at a design value. The P-wave velocity was found to vary from 910 m/sec to 1050 m/sec. For Poisson's ratio = 0.35 the shear wave velocity varied from 455 m/sec to 530 m/sec. The lower value was used, and the dynamic shear modulus of the soil was determined to be 2070 kg/cm^2 after incorporating the reductions recommended in WASH 1301[3]. The vertical-soil spring constant was determined to be K_y = 7661 MT/cm.[4]

Employing the concept[1] of dynamic load factor (DLF) and impulse and an assumed triangular load-time curve, the following general equation governs:

$$Y = Y_o \cos\ t + Y_o/\ (\sin\ t + Y_{st}\ \ (\ )\ \sin\ (t - \ )\ d \qquad (2)$$

DLF is defined as the ratio Y/Y_{st}, where $Y_{st} = F_1/K$, and F_1 = peak value of load on the load-time curve. Curves relating maximum DLF values to (t_d/T) ratios are given by Biggs[1] for various load applications where t_d = load duration and T = fundamental system period. For the assumed triangular load pulse with finite rise time, neither F_1 nor t_d are known, and reasonable estimates of t_d had to be made. Furthermore, the Biggs curves show that the maximum DLF occurs when the load duration is approximately equal to the natural period. Since our concern was with maximum response, the curves were used to search for the maximum dynamic effects. For an initial estimate of t_d = 0.06 sec, which was considered to be probable based on previous tests for smaller casks, Y_{max} = 0.61 cm.

It should be restated that the actual shape of the load-time curve is unknown, necessitating the assumption of the triangular load pulse with finite rise time. Within this assumption, several values of t_d were used with the Biggs curves in an attempt to bracket the response. The extreme case of t_d = 0.01 sec produced a maximum dynamic deflection of 1.14 cm. The corresponding peak spring force is 874.2 MT which produces a dynamic subgrade reaction on the pad base of 69.8 MT/m^2. This value is less than the design value of 122 MT/m^2. The energy absorbed by the soil spring varied from 1335 MT-cm to 4885 MT-cm for the previously stated assumed values of t_d. The kinetic energy of the cask is 1/2 mV^2 = 229,870 MT-cm. Within the stated limits, an estimate of the available energy for absorbtion by the cask is 99.4% for t_d = 0.06 sec and 97.9% for t_d = 0.01 sec.

The design details of the impact pad are described as follows:
Surface area: 4.88 by 7.92 m,
Bottom area: 9.75 by 12.8 m
Depth: 3.05 m
Pad weight: 670 MT
Transition steps: 4 at 0.81 m horizontal to 0.76 m vertical
Embedded steel armor plate: four 0.15-m layers at 2.44 by 5.5 m
Embedded hold-down bolts: 40 at 3.2 cm diam (ASTM A 307)
Armor plates floated on 0.15-m-thick high-strength grout
f_c concrete: 281 kg/cm^2
Reinforcing steel: ASTM A615 Grade 60

CONSTRUCTION

Construction of the drop pad started in April 1977 and was completed in June 1977. The initial site preparation consisted of removing the small, temporary drop pad that weighed 45 MT. p2600 The resulting hole in the ground was enlarged to 13 m long, 10 m wide, and 3 m deep.

After the initial layer of reinforcing steel was assembled on the base of the hole, a 15-cm layer of concrete was poured on the base to support the reinforcing steel. The balance of the reinforcing steel was assembled and welded in place. Fourteen metric tons of reinforcing steel were used in the installation. Forty 3.18-cm-diam ASTM A307 anchor bolts for the laminated steel armor plates were placed for embedment in the concrete. Forms were erected for the steps of the pyramid, and each step or layer of concrete was poured and allowed to set (Fig. 3).

Prior to pouring the final step of the pyramid, the steel armor plates were installed to permit any necssary in-place adjustments. Prior to setting the bottom plate in place, a 15 cm layer of quick-setting, high-strength grout was poured on which to float the plate and provide a transition medium for the high compressive stresses. The balance of the laminations were also bonded with grout (Fig. 4).

REFERENCES

1. J. M. Biggs, Introduction to Structural Dynamics, McGraw-Hill, New York, 1964.

2. F. E. Richart Jr., J. R. Hall Jr., and R. D. Woods, Vibration of Soils and Foundations, Prentice-Hall, Englewood Cliffs, N. J., 1970.

3. Directorate of Regulatory Standards, Classifications, Engineering Properties and Field Exploration of Soils, Intact Rock and In Site Rock Masses, WASH-1301(1974)

4. R. V. Whitman, "Analysis of Foundation Vibrations," in Vibrations in Civil Engineering, Butterworths, London, 1966.

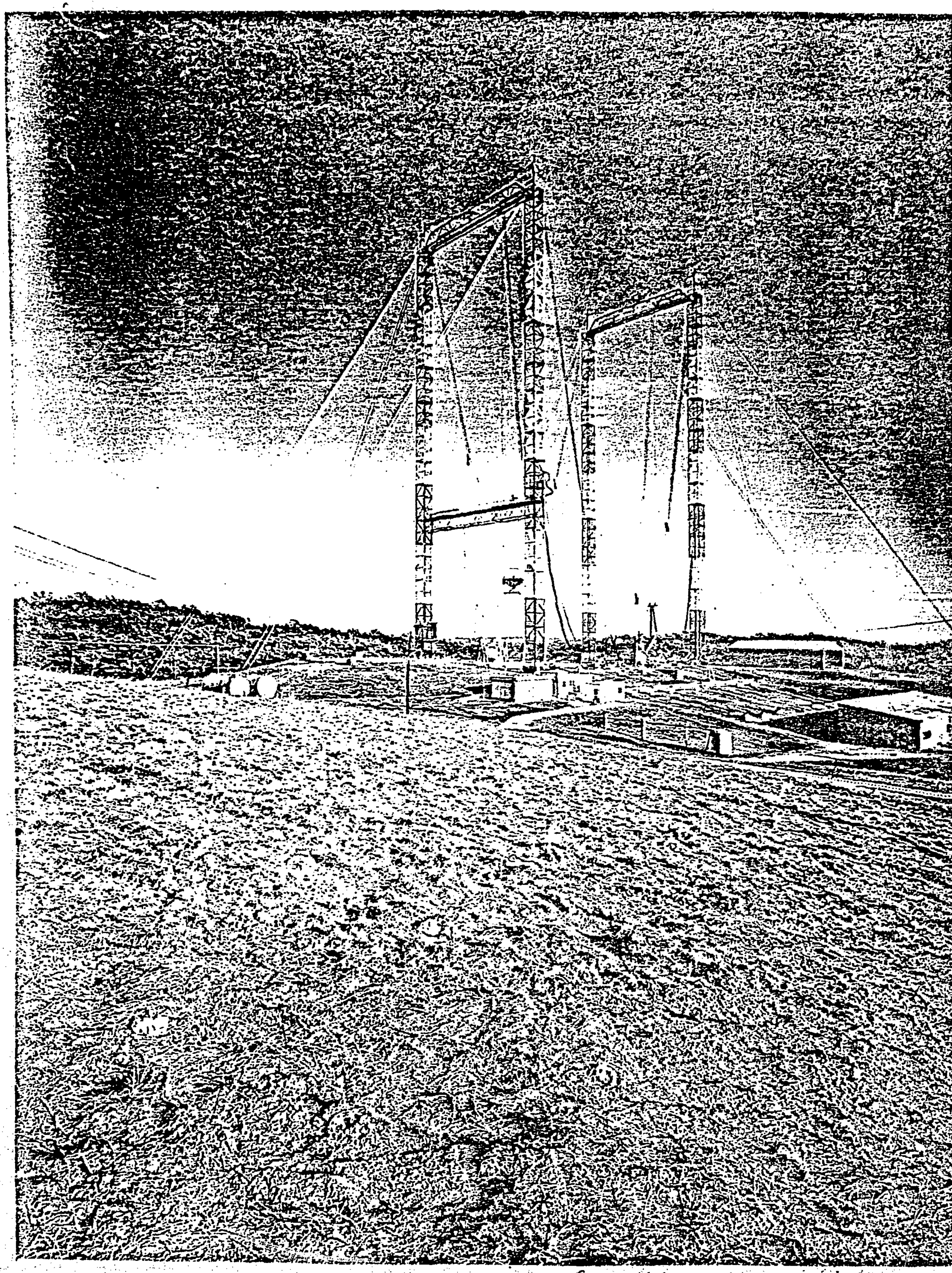
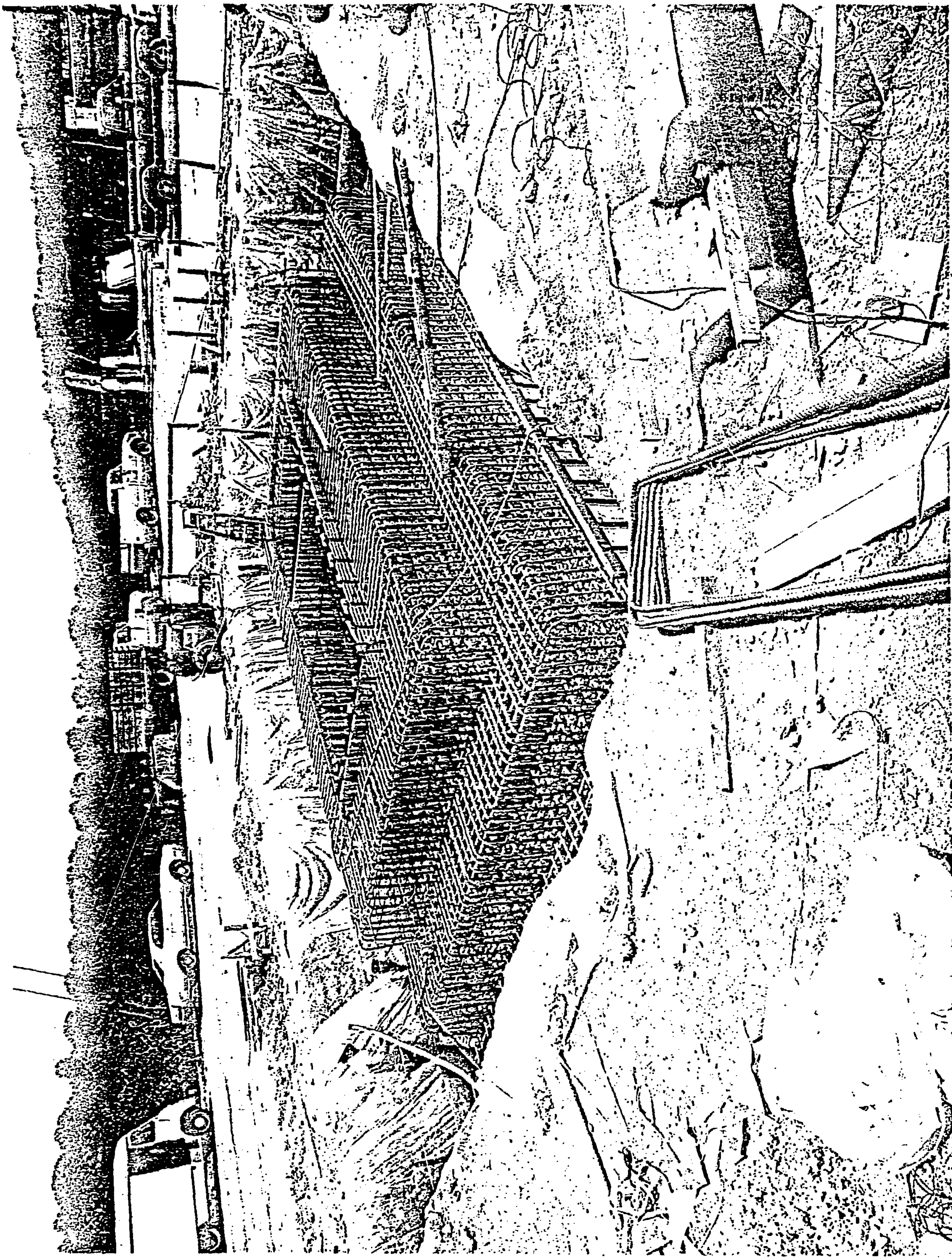


Fig. 1 Tower Shielding Facility



GDA.

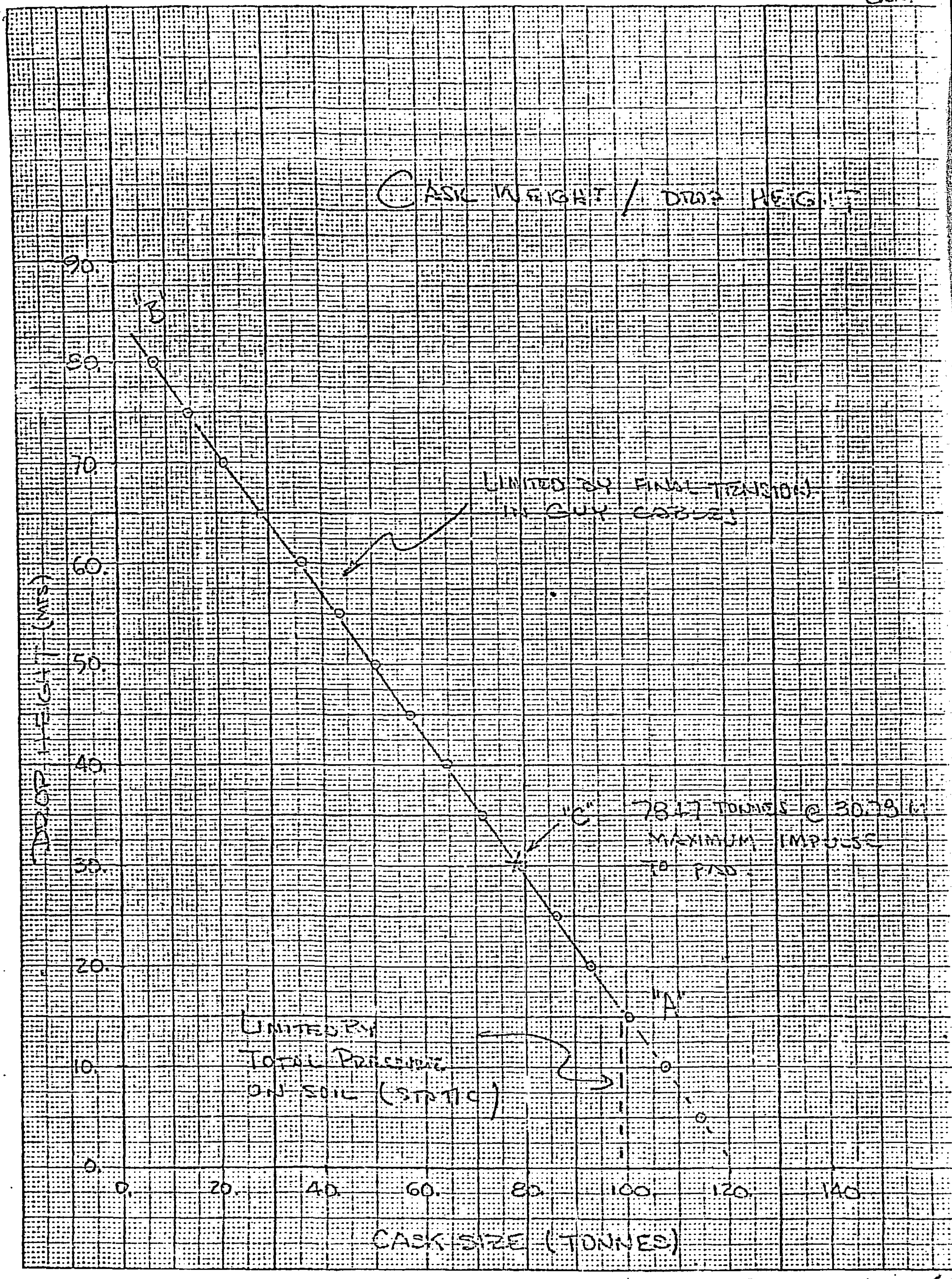


Fig. 2 Cask Weight vs Drop Height

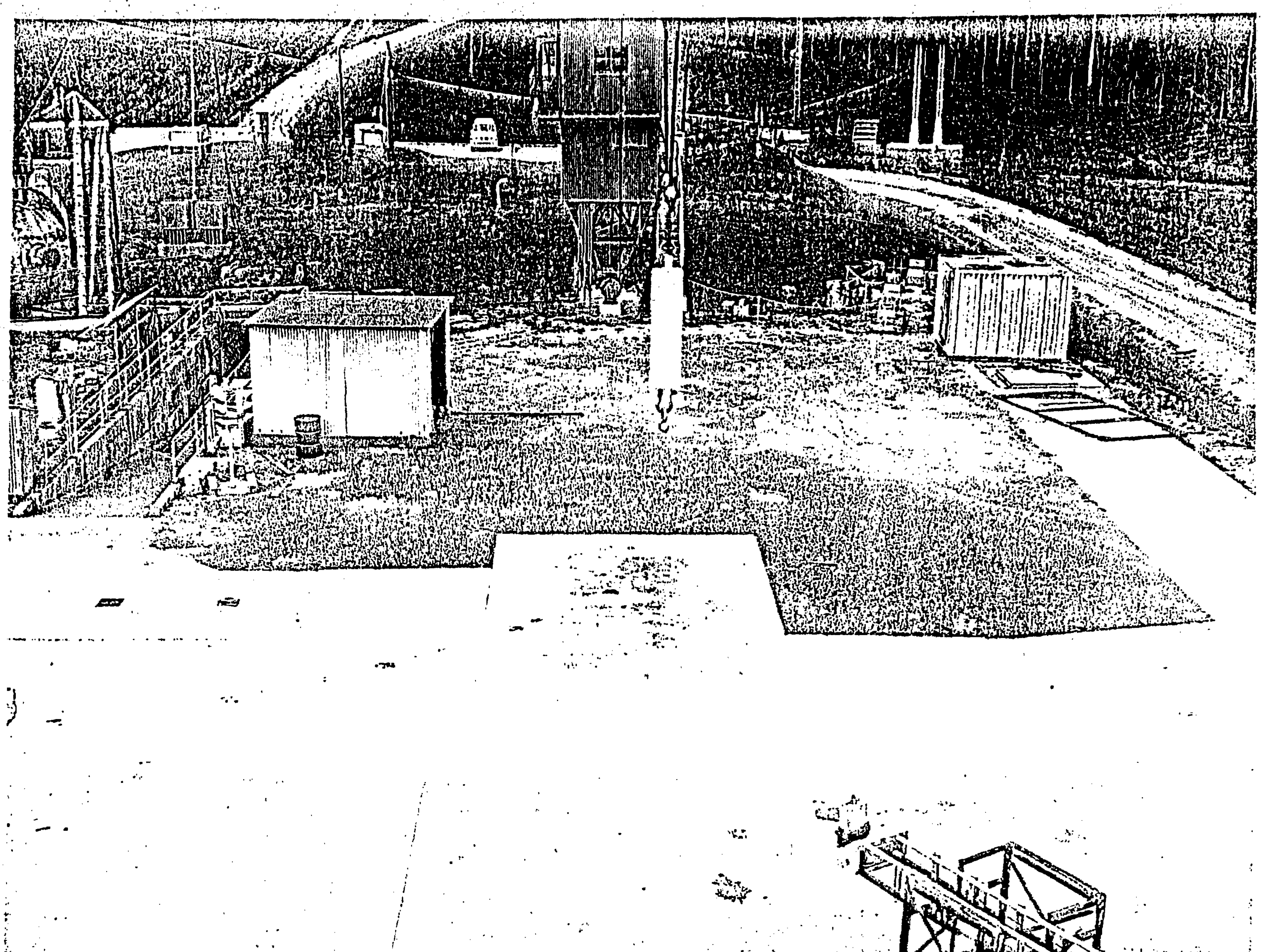


Fig. 4 Completed Outf– [illegible]