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Challenges in design and construction of building housing 100 T shake table

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Abstract— A shake table of 100 MT, largest in India was established at Indira Gandhi Center for Atomic Research, Kalpakam for conducting seismic qualification experiments of large size components of Fast Breeder Reactor. The table is placed on massive block foundation. The site is collocated with major safety structures. Attenuation relations were evaluated for rock blasting to avoid blasting related damage to the structures and personal. This paper presents the details of design and construction methodology adopted for this structure.

Index Terms— Control Blasting, Heat of hydration, shake table, shear wave velocity, reactor , water proofing, mass concrete

1 INTRODUCTION

INDIA has planned to construct four fast breeder reactors (FBRs), following Prototype fast breeder reactor (PFBR). For these FBRs, it is necessary to do further research in various areas of structural mechanics for enhancing safety and improving economy. Design of major components of PFBR and FBRs are controlled by the seismic loading. Seismic design should address many issues such as non-linear sloshing, strong fluid-structure interactions, non-linear random vibration of core subassemblies, nonlinear contact mechanics between grid-plate and core-support structure, strong interaction between top-shield structure with the cold pool structure through main vessel and sodium and dynamic buckling. Numerical simulation of these complex phenomena calls for extensive experimental validation. Considering many non-linearities, tests on larger scales are essential. Minimum 1/4 to 1/3 scale models can depict the phenomenon with reasonable accuracy. With this objective, structural dynamics lab in IGCAR is built for conducting seismic qualification experiments of large size components of FBR. The table is 6mx6m with a central hole of 3.5 m diameter. This will be a unique facility in the country. The capacity of the seismic shake table is 100 MT, largest in India, with six degrees of freedom and necessary data acquisition for simulation of earthquake ground motions and analysis. This special design gives the flexibility of testing large diameter vessels in hanging condition eliminating the requirement of stiff support structure reducing overall weight of pay load.

The building integrates the experimental areas along with control room, power pack area; other office area etc., at the same time isolates the office structure from vibrations generated in the experimental area. The lab is a RC framed structure of 20m x 40m and height 16.5m to facilitate tests of tall components. Steel tubular roof truss is provided for flexibility of top loading of specimen if need arises. There are tall openings with sliding door and rolling shutter for truck entry. The power pack room is provided with acoustic wall paneling. The experimental hall is also provided with 20MT capacity EOT crane for handling test components.

2.0 FOUNDATION

In order to attenuate the transmitting vibration to around 0.01g at 10 m from foundation the actuators (4 vertical and 4 horizontal) are placed on massive block foundation. Geotechnical investigations showed presence of weathered rock at 4 m depth (figure 1) from natural ground level. Shear wave velocity of the subsoil was evaluated through cross hole tests. Figure 2 shows the modulus of subsoil evaluated from shear wave velocity. Shake table is founded on massive block foundation on rock 7 m deep.

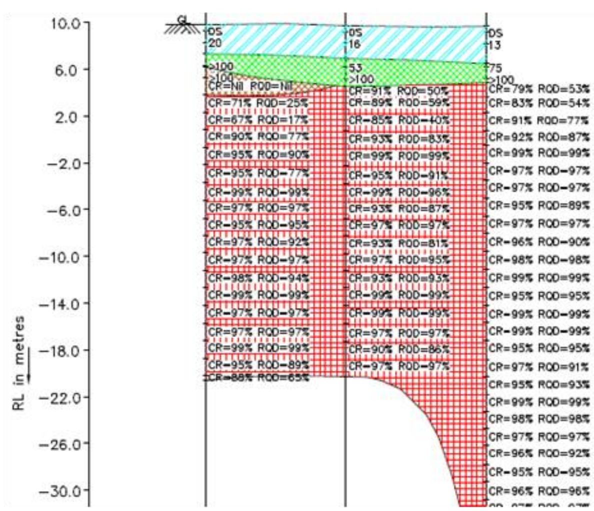

Fig. 1. Sub soil profile

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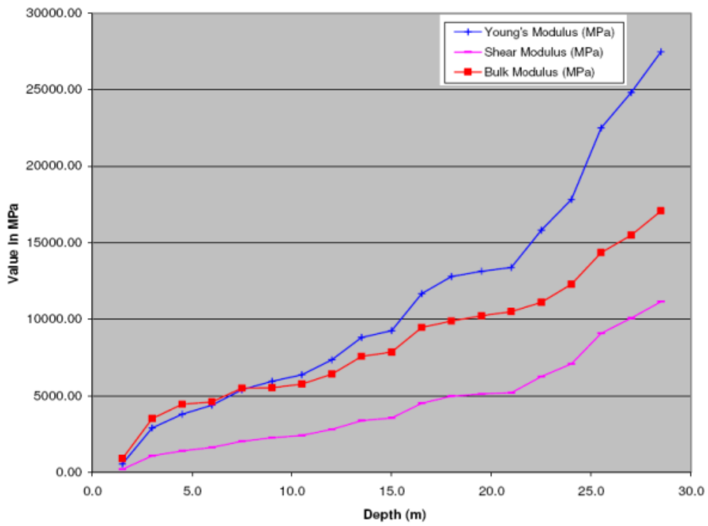

Fig. 2. Shear wave velocity profile

2.1 Control Blasting

The site is collocated with major safety structures. Excavation of hard rock by traditional method of drilling and blasting, is commonly associated with several unwanted effects like ground vibration, air blast and fly rocks. In order to maintain safety of the personal and structures, control blasting was adopted. Also, if ground vibration exceeds certain limits, it may cause damage to nearby structures and installations. Therefore, to ensure the safety, after necessary experimental studies by CWPRS procedure for control blasting was evolved.

2.1.1 Details of experimental studies

The rock formations at the site are Charnokite. The overburden consists of dense sand layer. A total of ten experimental blasts were conducted at the site . The charge weight per delay varied from 0.5 kg to 11.2 kg. The ground vibrations generated from the experimental blasts at Kalpakkam on rock and overburden were recorded at different distances. The resultant peak particle velocity , V_P is computed by the pseudo vector sum methods as follows .

$$V_P = \sqrt{V_T^2 + V_V^2 + V_L^2} \quad (1)$$

V_T- Transverse, V_V- vertical, V_L- Longitudinal components.

The amplitude and frequency of the elastic waves generated from blasting attenuates with distance. In addition the attenuation is also controlled by several other parameters like the quantity of explosive and properties of the transmitting rock mass. Therefore to predict the peak particle velocity at various distance from blast it is essential to determine the attenuation laws for each site , because the attenuation characteristics in an area, in general, is highly site specific. Equation 2 is used widely to study attenuation of blast vibration

$$V_P = K\left(\frac{R}{Q^{\alpha}}\right)^{-\beta} \quad (2)$$

V_P is the peak particle velocity (mm/s), R is the distance between observation and blast point, and Q is the quantity (kg) of explosive used per delay, and K, α, β are site specific parameters .

α, is the scaling parameters and square root scaling is used widely for prediction the blast vibrations [1],[2]. It is based on the assumption that the explosive charge is distributed in a cylindrical hole. K, and β depend on largely on type of rock and are determined for the site by carrying out trial blasts with varying weights (Q) and recording resultant velocity at different distances (R). Following attenuation relations were derived for over burden soil and rock surfaces from the least square and 95 % confidence level from figure 3 and 4.

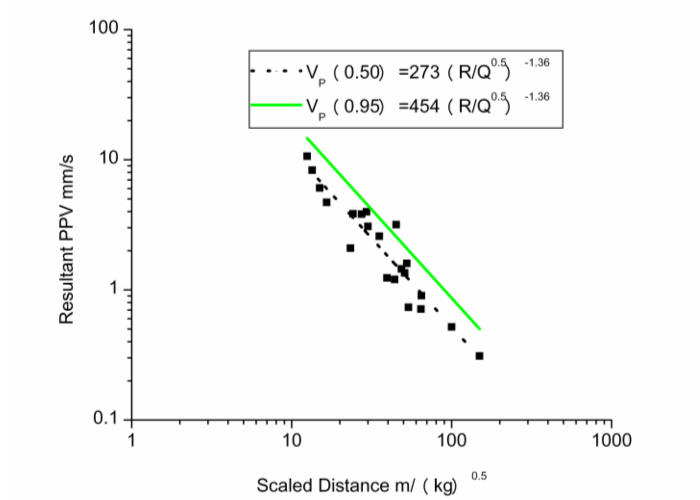

Fig. 3 Attenuation relation for Over burden

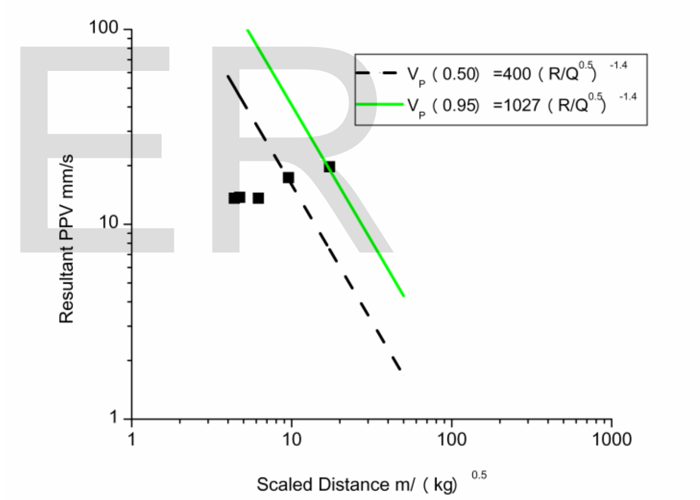

Fig. 4 Attenuation relation for Rock

$$V_{P(0.50)}(Over\ burden) = 273\left(\frac{R}{Q^{05}}\right)^{-1.36} \quad (3)$$

$$V_{P(0.95)}(Over\ burden) = 454\left(\frac{R}{Q^{05}}\right)^{-1.36} \quad (4)$$

$$V_{P(0.50)}(Rock) = 400\left(\frac{R}{Q^{05}}\right)^{-1.40} \quad (5)$$

$$V_{P(0.50)}(Rock) = 1027\left(\frac{R}{Q^{05}}\right)^{-1.40} \quad (6)$$

So breaking of 1550 m^3 of rock using control blasting was carried out without exceeding the permitted peak particle velocity of 8 mm/s, based on above equations. The procedure adopted for controlled blasting operation was line drilling method. The diameter of the blast hole was 32 mm and depth ranged from 0.75 m to 1.5 m. Blast material used was

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gelatin gel each 125g m.The charge factor was maintained approximately between 0.40 to 0.50 kg/m^3 of rock so as to make fragmentation only in the excavation pit (figure 5). The peak particle velocity was measured during actual blasting.

Fig. 5. Control blasting layout

2.2 Water proofing

Shallow water table in site called for an effective water proofing system. In order to isolate seismic mass from building there is a gap of 25 mm. So complete water proofing was essential. Bentonite geotextile waterproofing with integrated polyethylene liner was used.

In this system the high swelling, low permeable sodium bentonite is encapsulated between the two geotextiles. A proprietary needle punch process interlocks the geotextiles together forming an extremely strong composite that maintains the equal coverage of bentonite, as well as, protects it from inclement weather and construction related damage. Once backfilled, it forms a monolithic waterproofing membrane by forming a low permeability membrane upon contact with water. When wetted, unconfined bentonite can swell up to 15 times its dry volume. Whenconfined under pressure the swell is controlled, forming a dense, impervious waterproofing membrane. This swelling

Fig. 6 Water proofing

action will self-seal small concrete cracks caused by ground settlement, concrete shrinkage, or seismic action. Figure 6 shows the water proofing works at this site.

3. DESIGN OF FOUNDATION BLOCK

The seismic block was provided with counter fort retaining wall to achieve vibration isolation (figure.7). The seismic mass block concrete was 1043 m^3 and could not be done in single layer due to high heat of hydration. Temperature rise for M35 grade concrete with 400 kg/m3 OPC, was analysed using ACI 207.2R-95 [3]. Number and size of pours was determined so as to avoid joints at critical locations ([4], [5].

Fig. 7 Retaining walls details

Hence concrete was placed in four pours with maximum single pour of 297 $m^{3.}$ Temperature controlled concrete with a placement temperature less than 23ºC, was achieved using 60-90% ice flakes ,along with chilled water. Temperature reinforcement of 16mm dia @ 200 centers was placed in each pour.

In order to transfer the heavy axial and torsional reactions from powerful actuators a steel frame work with heavy embedded parts were also embedded well within concrete (figure8 and 9).

Fig. 8 Supporting steel structure

A smaller shake table of 10MT capacity will also be founded on the same block (figure 9).

3.1 Loading environment

In addition to self weight and earth pressure following load cases were considered in the analysis of the mass block.

3.1.1 Live load

A live load of 10 KN/m2 for maintenance has been considered.

3.1.2Pay Load

A total load of 1500 KN was assumed to act at the jack position. 20 % of jack load assumed to act for smaller actuator with $\pm 15^{O}$ deviations.

3.1.3 Seismic load

The structure is in Zone III. Lateral loads have been computed based on IS 1893:2002 . Earthquake loads have been considered in all three directions with suitable load combinatios.

3.1.4 Actuator load

There are two types of actuators. Main and subsidiary are identified as AL and SAL. The load taken as harmonic time varying load. The maximum stress was considered after running the programme at various frequencies.

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3.2 Mathematical Model.

The foundation was modeled with solid element of approximate size 0.775 x 0.96x 0.75 m . Corners and at the points of loading smaller element sizes have been used to represent geometry (figure 9) as nearly as possible. The whole foundation model was discretised in to 2547 elements. They were connected with 3408 nodes each with six degrees of freedom.

Fig. 9 Foundation block of shake table before erection

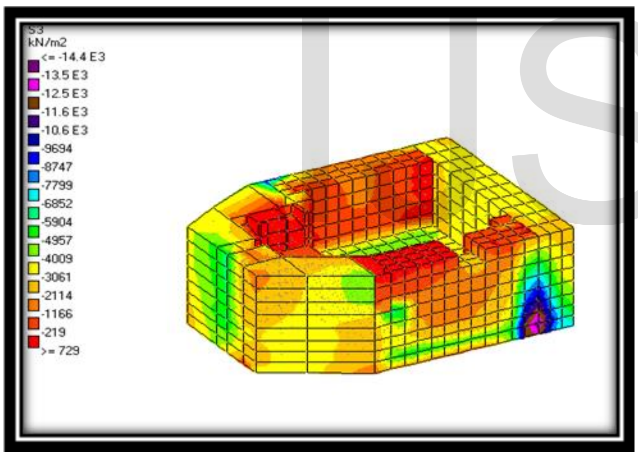

Fig. 10 Maximum Principal Stress for the foundation

Maximum principal stress variation of 3D idelised model is shown in figure 10.

3.3 Material Properties

M 35 grade concrete and Fe 500 steel was used. The design was based on IS 456: 200.

3.4 Results discussion

The foundation was analyzed for limit state of collapse and limit state of serviceability and maximum values of stress under all load combinations at different levels were extracted. Maximum deflection as per analysis was < 1mm and negligible. Reinforcement was provided for resisting the stress in three mutually perpendicular directions. Based on Wood and Armor equations, the shear stress was added to normal stress. After determining maximum stress it was converted to equivalent force by multiplying element size along that plane. This was applied to all critical elements. Figure 11 shows the final stage of shake table foundation before erection of shake table.

Fig. 11. Final stage before erection of shake table

4. CONCLUSION

Design and construction of foundation and the building for 100 T shake table was completed successfully within 12 months. Heat of hydration due to mass concreting was taken care in the design. Proximity to the safety structures called for a detailed study on rock blasting and attenuation relations. It was evaluated for the rock present at the site before actual excavations. Complete water proofing system was placed to avoid any leakage of water.

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