Design of Foundations for NPP Structures

S M Palekar[1)] , K V Subramanian [1)] A Shrivastava[2)], U S P Verma[2)] and A S Warudkar [(2)]

1) TCE Consulting Engineers Limited, Mumbai, India.
2) Nuclear Power Corporation of India Limited, Mumbai, India.

ABSTRACT

The purpose of the foundations is to provide a stable base for the superstructure and to transfer safely all loads and their effects to the ground. To achieve this purpose, it should have adequate factor of safety against sliding, overturning and floatation and the maximum bearing pressure under any point of foundation should be within the allowable limit. The simultaneous occurrence of different load conditions of downward gravity loads, lateral and vertical loads due to seismic event and upward buoyancy effect due to sub-soil water in various combinations should be considered to generate most critical design condition. Under the most critical design condition, the foundation in all probable cases partially loses contact with the substrata. The analysis and design of foundations for the most critical design conditions are discussed in this paper with an illustrative example and the solution adopted for a Nuclear Power Plant structure under construction in India.

INTRODUCTION

NPP structures The design of foundations is governed by the sub-soil conditions available at the specific location of the structure. In case of, it is, in addition, governed by the nuclear safety considerations. These involve radiological shielding requirements, structural integrity of the structure and the leak-tightness or the containment of the fluids inside the Nuclear Power Plant (NPP) structure. The containment of the fluids in a leak-tight manner is required for a very long period of time much beyond the operative life span of the plant. The structural integrity should be established considering all the severe and extreme environmental effects and the abnormal effects that may take place during that period.

In this paper, analysis and design of foundation of a NPP structure under construction in India is discussed. The structure is subjected to Dead loads, Imposed variable loads, Equipment loads, Thermal and Pressures effects under Test, Operating and Accident conditions. The structure is also subjected to Static earth pressures, dynamic earth pressures from outside, hydrostatic pressures and hydrodynamic pressures from inside and effects under Operating Basis Earthquake (OBE) and Safe Shutdown Earthquake (SSE) conditions. The reaction of all these are transferred to the foundation. In addition, the foundation is subjected to the overburden of subsoil and upward water pressure due to buoyancy. The transfer of superstructure reactions to the foundation under static conditions is a simple matter unlike superstructure reactions under SSE or OBE conditions. Time History analysis or the Response Spectrum analysis is necessary to obtain effects of SSE and OBE. In the process of carrying out Mode combinations and combination of three spatial components the signs of the stress resultants is lost. Special algorithms are employed to attach signs to the superstructure reactions obtained under SSE or OBE conditions. [1].

DESIGN PARAMETERS

Apart from structural integrity, the foundation should be able to transfer all the forces to the sub-soil in an effective manner. It should have sufficient resistance to sliding, overturning and floatation. Under seismic conditions and high water table, foundations tend to be critical to sliding, overturning and floatation. It is recommended in USNRC SRP 3.8.5 [2], for highest water table, factor of safety (FOS) against sliding, overturning and floatation shall be 1.5 for OBE condition and 1.1 for SSE condition. In case of inadequate FOS, dead weight on the foundation can be increased to improve stability against overturning and floatation to some extent. Other alternatives are discussed more in detail later. For sliding resistance, shear key of adequate dimensions can be provided. An optimum size of the foundation is arrived at to achieve above FOS.

Further, the sub-soil should have sufficient strength to withstand the pressures exerted by the foundation without excessive deformations. The pressure under a foundation is directly related to the applied loads and the size of foundation. The load transfer takes place through bearing action only at soil- foundation interface. In the event of very high upward water pressure and very high overturning moment and relatively low stabilising load the foundations loose contact with the sub-strata. The pressures are to be calculated considering the actual contact area of foundation with the substrata. To satisfy the maximum allowable bearing condition, an optimum size of the foundation can be worked out.

Satisfying maximum allowable bearing pressure as well as having adequate FOS against sliding, overturning and flotation completes sizing of foundation. Among these parameters, allowable bearing pressure, loss of contact with the substrata and FOS against overturning are related to each other. As the loss of contact with substrata increases, the bearing

pressures rise and the FOS against overturning reduces. The codes on foundation design, normally, specify only two constraints i.e. allowable bearing pressures and the maximum FOS against overturning.

RELATION BETWEEN DESIGN PARAMETERS

Most of the foundations for various buildings fall in the category of rigid foundations. For such rigid foundations, rectangular and circular in shape, variation in factors of safety against overturning is studied with respect to the loss of contact, the eccentricity of axial load and resultant bearing pressure. [3]

Rectangular Foundations

Table-1 presents lift-off coefficient, bearing pressure coefficient and factor of safety against overturning for various values of eccentricities for a rigid rectangular footing subjected to axial load and Uni-axial bending moment. With reference to Table-1, it is observed that if 13.3 times average bearing pressure is less than allowable bearing pressure, then 85 % loss of contact may be allowed since factor of safety against overturning is still higher than 1.1. Though 85% is a very high theoretical figure, a practical limit on factor of safety may be considered as 1.33 allowing loss of contact of the order of 62.5%. Any limit on loss of contact less than equal to 50% would lead to factor of safety higher than 1.5, which is very stringent for SSE effects since this value is allowable under OBE.

Table– 1

Loss of Contact for Rigid Rectangular Footing Subjected
To Axial Load & Uni-Axial Moment

SR. NO.	ECCENTRICITY COEFFICIENT	CONTACT AREA COEFFICIENT	LIFT-OFF COEFFICIENT	BEARING PRESSURE COEFFICIENT	FACTOR OF SAFETY AGAINST OVERTURNING	REMARKS
1	0.167	1.000	0.000	2.000	3.000	
2	0.200	0.900	0.100	2.222	2.500	
3	0.250	0.750	0.250	2.667	2.000	
4	0.300	0.600	0.400	3.333	1.667	
5	0.333	0.500	0.500	4.000	1.500	OBE
6	0.350	0.450	0.550	4.444	1.429	
7	0.375	0.375	0.625	5.333	1.333	
8	0.400	0.300	0.700	6.667	1.250	
9	0.450	0.150	0.850	13.333	1.111	SSE

Legend:

Footing size : Length = L, Breadth = B
Axial Load = P
Eccentricity = eb, Bending Moment = P x eb
Eccentricity Coefficient = Ce = Eccentricity / Breadth = eb / B
Contact Area Coefficient = Cac = Contact length / Breadth = Lbc / B
Lift-off Area Coefficient = Cla = 1 - Contact Area Coefficient = 1 - Cac
Bearing Pressure Coefficient = Cbp = Max. Bearing Pressure / Avg. Bearing Pressure
= Pr(max.) / Pr(avg.)
Average Bearing Pressure = Pr(avg.) = P/ L x B

Circular Foundations

Table - 2 presents these parameters for rigid circular foundations subjected to axial loads and uni-axial bending moment. With a similar logic as above, 94% loss of contact may be allowed for a circular foundation, provided allowable bearing pressure is higher than 12.92 times average bearing pressure which satisfies factor of safety against overturning to be higher than 1.1 for SSE. Here again 94 % is a very high theoretical value; a practical limit of the order of 1.32 on factor of safety leads to 76% loss of contact. Any limit on loss of contact less than or equal to 63 % would lead to factor of safety higher than 1.5, which is very stringent for SSE since this value is applicable to OBE.

Table – 2

Loss of Contact for Rigid Circular Footing Subjected
To Axial Load & Uni-Axial Moment

SR. NO.	ECCENTRICITY COEFFICIENT	CONTACT AREA COEFFICIENT	LIFT-OFF COEFFICIENT	BEARING PRESSURE COEFFICIENT	FACTOR OF SAFETY AGAINST OVERTURNING	REMARKS
1	0.59	0.50	0.50	1.50	1.70	
2	0.66	0.39	0.61	1.95	1.52	
3	0.67	0.37	0.63	2.03	1.50	OBE
4	0.72	0.29	0.71	2.68	1.38	
5	0.76	0.24	0.76	3.23	1.32	
6	0.79	0.20	0.80	3.97	1.27	
7	0.85	0.12	0.88	6.46	1.18	
8	0.90	0.07	0.93	12.02	1.11	
9	0.91	0.06	0.94	12.92	1.10	SSE
10	0.94	0.03	0.97	27.45	1.06	
11	0.98	0.01	0.99	90.58	1.02	

Eccentricity Coefficient = e/R, where R is the radius of foundation. Other notations are self-explanatory.

The above tables provide guidelines for deciding the extent of loss of contact that may be considered in the design with reference to the FOS against overturning and the bearing pressure.

MEASURES TO RESTRAINT OVERTURNING

Reactor Building for a typical PHWR in India has Inner and Outer Containment Walls, Internal Structure and Calandria Vault supported on the Raft foundation. When analysed for all loading effects as discussed above, following parameters evolved for 62.0m diameter, 5.5m thick raft.

CASE	Maximum Bearing Pressure	Maximum Loss of Contact	Factor of safety against Overturning
No restraint for overturning	260.38 T/m^2	76.43 %	1.54

It is observed that adequate FOS against overturning is available and the maximum bearing pressure is within acceptable limit. To restrict the loss of contact following alternative measures were considered.

a) Consider the passive resistance offered by the rock surface along the side of 5.5m thick raft.

The resistance offered by the side rock is passive in nature. Under the action of an overturning moment when the rock starts loosing contact with the substrata below, it will lift up and slide past the side rock surface. Under this situation, the passive resistance of the side rock will play its role. Its magnitude will be proportional to the loss of contact. Rock mass was analysed separately to evaluate the resistance it offers for a unit displacement. Side rock resistance value obtained from analysis was reasonably scaled down considering the uncertainty involved and used for raft analysis. It was ensured that the maximum principal tensile stress (31.5 T/m2) in the rock do not exceed the permissible values (60 T/m2). The loss of contact and other parameters obtained are as follows:

CASE	Maximum Bearing Pressure	Maximum Loss of Contact	Maximum side rock resistance
Restraint by side rock resistance.	115.4 T/m^2	50.42 %	293.2 T/m i.e. 53.3 T/m^2

The values of Rock-Concrete bond in shear were actually measured at site by casting a block of concrete (1000 long x 750 wide x 700 high) within a trench cut into the rock. This block was pushed up by jacks. The resistance, it offers till failure is noted. The Rock-Concrete bond in shear measured at failure at site is 132.6 T/m2.

b) Provide prestressed rock anchors to clamp the Raft down to rock.

The prestressed rock anchors apply the resistance to the lift off situation in active manner. These rock anchors are considered all along the perimeter of the raft. Raft was analysed with rock anchors idealised using spring elements with a defined tension value. In all 200 nos. of rock anchors each having 27 nos. of 1/2inch strands with an allowable capacity of 250 T were used.

CASE	Maximum Bearing Pressure	Maximum Loss of Contact	Additional tension in rock anchor due to lift-off
Restraint by prestressed rock anchors	158.05 T/m^2	36.91 %	23.7 T

DISCUSSION

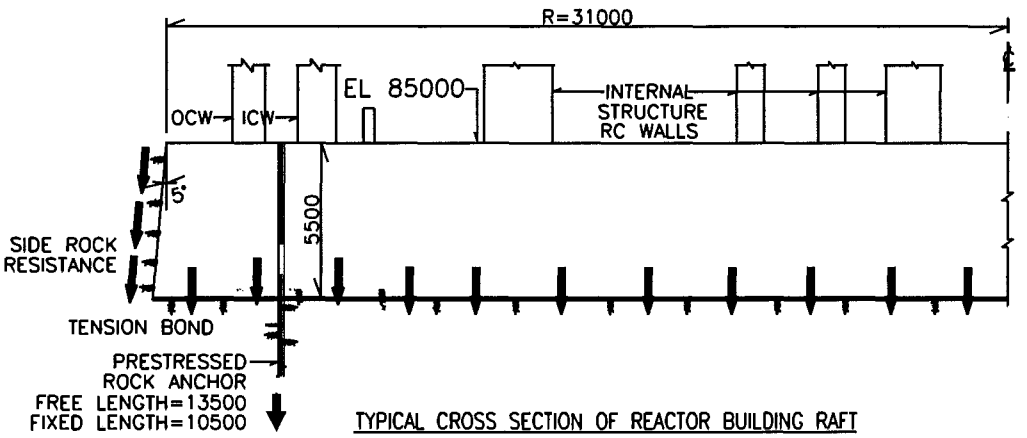
It is observed that to reduce loss of contact with substrata from 76.4 % to 36.9 %, total prestressing force of 50,000 T was required. Though it has little uncertainty about it, it is a very costly solution. The second alternative of side rock resistance though not costly, involves a cautious approach for rock cutting and developing a negative slope of 5^0 with the vertical so that under the lift-off situation, the raft will develop a wedge action with surrounding rock. This is possible only if the surrounding rock is solid and in sound condition and it remains undamaged during the rock excavation. This will be revealed only after excavation is over. At that stage, it may be too late to change the design and construction drawings. Even if all this is achieved, the loss of contact is not less than 50%. A lot of uncertainty is involved in this solution.

Other inexpensive method would be to consider tension bond value available between the rock and the raft bottom surface. In the past, experiments to measure the tension bond values at site have reported values varying from 3.5T/m2 to 34.9 T/m2 [4]. To estimate the tension bond value by experimentation, Concrete blocks were cast on the excavated rock surface at site. These blocks were lifted up till failure. From these experiments, tension bond failure values varying from 13.84 T/m2 to 31.86 T/m2 were obtained. Consideration of rock tension bond will improve stability reducing the lift-off of 76.43 % as obtained above.

CONCLUSION:

Analysis and design of foundations of NPP structures involve consideration of superstructure reactions, soil-structure-interaction, obtaining partial area of contact with substrata etc. The methods to limit partial loss of contact with substrata are discussed in this paper with an example. The stabilising effects of prestressed rock anchors, side rock resistance and the tension bond on the raft foundation are compared.

When faced with large loss of contact with substrata, alternatives that are available to reduce the loss of contact are the consideration of tension bond between the excavated rock surface and the concrete, bond between side rock and concrete and provision of prestressed rock anchors. The effects of these three alternatives cannot be added and they are to be considered effective only one at a time. As the foundation starts loosing contact with the substrata, the tension bond becomes effective. When tension bond capacity is overcome, the shear bond on the side surface becomes effective. Due to wedge action, the actual resistance available on side surface is larger than that due to only shear bond. In case of provision of prestressed rock anchors, both these options are ineffective. From economic considerations prestressed rock anchors should be considered only as a last resort.



TYPICAL CROSS SECTION OF REACTOR BUILDING RAFT

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