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Capacity evaluation of Lead cell foundation

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Abstract— Shielding of radiations emitted by radioactive specimens is usually done using lead wall as lead is considered to be the best radiation shielding metal. Impact toughness of irradiated samples made out of stainless steel, from the PFBR main vessel and safety vessel, is to be found out for design requirements. For this a shielded impact test facility is needed due to the high gamma emission from the irradiated specimen. Lead is well known among the heavy metals for its shielding properties. The present work is based on the evaluation of capacity of the existing foundation for static load due to the placement of lead bricks as shielding arrangement in both strength and serviceability.

Index Terms—Base pressure, Foundation, Interaction curve, Lead cells, Modulus of subgrade reaction, Radiation shielding, Shell and wall finite elements, Stress resultants, Wood - Armer technique.

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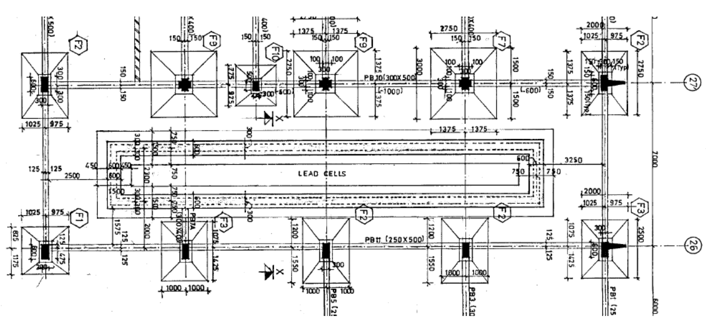
1 INTRODUCTION

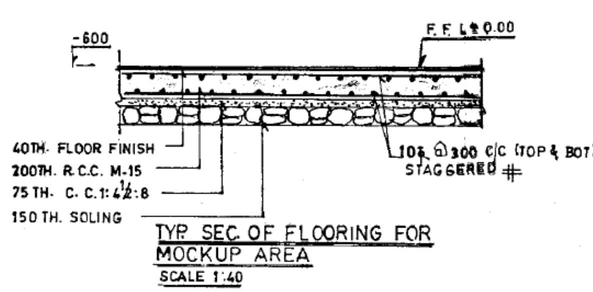
THE foundation for the lead cell shielding in RML (Radio metallurgy Laboratory), Indira Gandhi Center for Atomic Research (IGCAR), Kalpakkam, was designed and constructed 25 years ago as shown in the Fig-1, considering the loading due to lead cell mock up arrangement. The experimental setup is such that a central impact testing equipment will be installed with a block foundation and impact test will be conducted on irradiated specimen of dimensions 55 mm length and 5 sq.mm cross sectional area. The test specimen encased in a capsule is irradiated in FBTR (as PFBR is yet to be commissioned) with 100 neutrons, which is comparable to 40 year irradiation in the actual main vessel, safety vessel stainless steel material in PFBR. The irradiated specimen has high gamma emission dose and therefore the whole setup with provision for inspection and maintenance on all sides and top had to be shielded from radiation. The impact toughness of the irradiated specimen is measured in a standard impact testing machine, with an impact velocity of 5 m/s, as this is a very important design parameter. The change in the impact toughness after irradiation is to be measured. Based on the strength of radiation that will be developed, the shield material thickness and height were decided as 0.25 mm and 3.1 m respectively with 0.12 mm thick stainless steel roof shield. Due to the proximity to the safety building, excavation and reconstruction of the existing foundation was not possible. So it was decided to evaluate the capacity of the existing foundation for the current requirement. The foundation was modeled in NISA /CIVIL package of NISA suit of software with the help of shell elements. The boundary conditions used in this model are foundation spring elements generated based on the modulus of sub-grade reaction determined theoretically for the soil.

2 PROBLEM DEFENITION AND LOCATION

The existing foundation is a composite system comprising of RC walls, top slab and base slab. The top slab is continuous and covers the opening created by the RC walls on all four sides and the base slab forms the strip footing for the RC walls. It was probably intended at the time of designing that, the lead shield would be placed on the RC wall location such that the centerline of the lead shield and that of the wall coincide and the entire load is transferred to the foundation through the walls. But, due to experimental requirements, there is an offset of 75 mm as shown in the fig (2) in the placement of the lead bricks and that the lead cells needed to be placed on the top slab below which there is no RC wall. The foundation is taken to a depth of 2500 mm where the safe bearing capacity of soil is 250 KN/m^2. Below the top slab the soil is well compacted and is comprised of layers of PCC, soling and gravel filling. The other parts of the foundation are filled up with soil. This foundation is to be evaluated for its various structural capacities, against the static loading due to lead cell shield. The structural effects of arrangement of lead shielding of the specified height and thickness on existing foundation is studied and the strength of the foundation against the loading is evaluated in this paper. The following figures Fig-1 (a), (b), (c) and Fig-2 (a), (b) show the location of the lead cell foundation and shielding:



(a)



(b)

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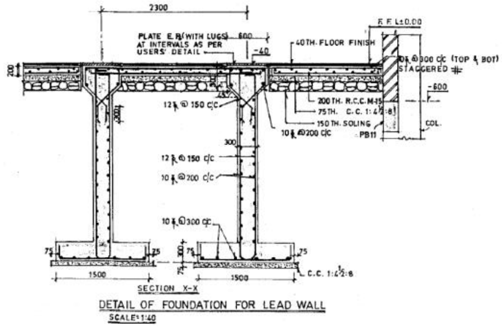
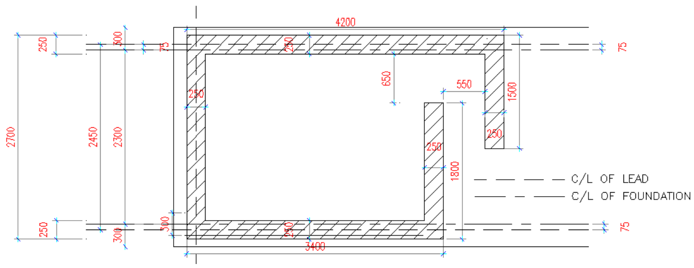
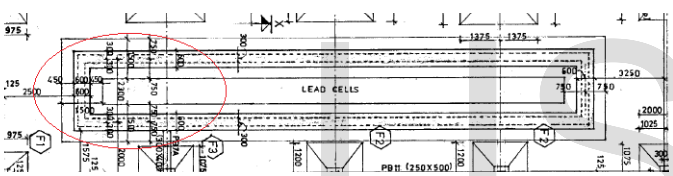


(c)
Fig-1 Foundation details as per drawing no. IGCAR/RML/0202/REV1



(a)



(b)
Fig-2 Arrangement of Lead wall over the foundation

3 ANALYSIS

Static analysis was done in NISA for the dead loads (DL) due to the shielding arrangement.

3.1 Concrete material properties

Grade of concrete is M-20, Poisson's ratio is 0.2 and modulus of elasticity is 22360 N/mm^2

3.2 Soil Properties

In order to idealize soil, Vesic' s modulus of sub-grade reaction [4] was considered and following soil properties were assumed in the model:
Modulus of elasticity of the soil, E_s = 150 N/mm^2 [4]
Poisson's ratio of soil, μ = 0.3
Width of the foundation, B_f = 1500 mm
Thickness of the base raft, t_f = 300 mm
Moment of inertia of foundation section, $I_f = \frac{B_f t_f^3}{12}$ = 3.375E+09 mm^4
Modulus of elasticity of the foundation, $E_f = 5000\sqrt{f_{ck}}$ = 22360.68 N/mm^2
Modulus of sub-grade reaction, $K = 0.65\sqrt[12]{\frac{E_S b^4}{E_b I}}\cdot\frac{E_S}{1-\mu^2}$ …….(1)

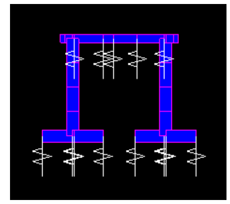


Fig-3 Mathematical model of the lead cell foundation

Base raft and wall elements were modeled with 3D general shell element, 300 mm thick. Top slab was modeled with 3D general shell element, 200 mm thick. In NISA CIVIL, foundation springs (NKTP = 38, NORDER = 1) were generated by specifying the vertical modulus of sub-grade reaction based on the equation no (1).

3.4 Lead cell loads

The height of the wall is 3.1m and it is 0.25 m thick. Stainless steel roofing of 0.12 m thick is also used. The pressure due to lead cell loading = (114*3.1) = 353.4 KN/m^2 where 114 KN/m^3 is the density of lead. Additional pressure of 33.912 KN/m^2 due to self weight of the roof was also considered. Total pressure applied in the model was 390 KN/m^2

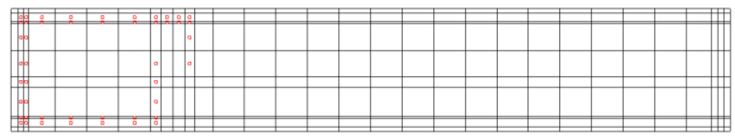


Fig-4 Plan of top slab loaded with lead bricks

4 ANALYSIS RESULTS

4.1 Nodal displacements

For the structural stability of the lead shield arrangement the maximum permissible displacement of the system after the placement of shield is to be less than 1 mm. Maximum vertical nodal displacement = 0.0227165 mm (downwards). The maximum values of the 3 translational DOFs and the nodes at which they occur respectively are listed below and the figure shows the location of the respective nodes:

Table-1 Maximum displacements

DOF	UX	UY	UZ
Value, mm	0.009508	-0.02272	-0.0464
Node no.	657	667	691

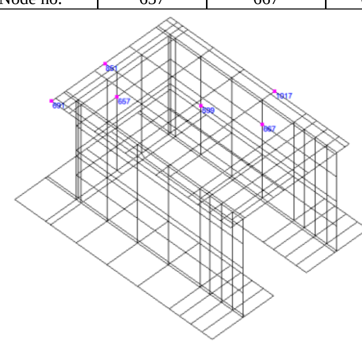


Fig-5 Points where maximum nodal displacements/rotations occur

4.2 Plate bending moments

The plate elements for which the maximum and minimum stress resultant values occurred were considered critical and the final moments were determined based on Wood's criterion [2] and is listed below:

Table-2 Plate bending moments

TYPE OF ELEMENT	ELEMENT ID	NODE ID	ANALYSIS RESULTS IN N mm/mm			FINAL MOMENTS in KN m/m (Wood's criteria)			
						Bottom		Top	
			MXX	MYY	MXY	Mux	Muy	Mux	Muy
SLAB	916	950	-1277.43	1825.43	-804.998	0	2.332716	-1.63243	0
SLAB	912	918	-1737.77	839.14	999.987	0	1.414575	-2.73776	-0.16085
WALL	666	663	-3944.69	-1659.44	227.402	0	0	-4.17209	-1.88684
WALL	629	628	4863.04	-1629.78	-314.793	4.923842	0	0	-1.65016
SLAB	920	923	-974.193	4523.7	209.588	0	4.568791	-0.9839	0
SLAB	942	980	-1821.12	736.135	1411.16	0	1.829623	-3.23228	-0.67503
SLAB	760	790	-1489.99	1908.66	804.495	0	2.343034	-1.82908	0
SLAB	756	758	-1933.17	1612.56	-831.302	0	1.970037	-2.36172	0
WALL	664	663	-3248.59	-2447.96	-92.5892	0	0	-3.34118	-2.54055
WALL	627	625	3960.78	1627.54	-234.487	3.994564	1.641422	0	0
WALL	588	559	811.105	-2017.61	-96.2744	0.815699	0	0	-2.02904
WALL	668	666	-4897.99	1595.92	355.689	0	1.62175	-4.97726	0
WALL	598	663	1079.57	-3313.59	-256.229	1.099383	0	0	-3.3744
SLAB	725	724	-1808.18	727.714	-1402.5	0	1.815552	-3.21068	-0.67479

4.3 Plate normal forces

The plate elements for which the maximum and minimum stress resultant values occurred were considered critical and the final normal forces were determined based on Wood's criterion [2] and are listed below:

Table -3 Plate normal forces

TYPE OF ELEMENT	ELEMENT ID	NODE ID	ANALYSIS RESULTS IN N mm/mm			FINAL FORCES in KN/m			
						Tensile forces		Compressive forces	
			NXX	NYY	NXY	Nux	Nuy	Nux	Nuy
SLAB	916	950	25.7032	8.59296	3.22006	0.028923	0.011813	0	0
SLAB	912	918	25.1535	29.0153	-7.95911	0.033113	0.036974	0	0
WALL	666	663	-34.0251	-75.6113	25.5913	0	0	-0.05962	-0.1012
WALL	629	628	-7.52095	-11.5637	-4.08597	0	0	-0.01161	-0.01565
SLAB	920	923	4.08352	0.81664	0.235578	0.004319	0.001052	0	0
SLAB	942	980	3.60596	2.51689	2.93936	0.006545	0.005456	0	-0.00042
SLAB	760	790	24.9855	4.99759	-3.0058	0.027991	0.008003	0	0
SLAB	756	758	13.3292	26.6518	7.9029	0.021232	0.034555	0	0
WALL	664	663	-34.0974	-75.6257	1.73051	0	0	-0.03583	-0.07736
WALL	627	625	-34.0108	-75.633	25.4477	0	0	-0.05946	-0.10108
WALL	588	559	1.91116	-8.86884	-12.1994	0.014111	0.003331	-0.01029	-0.02107
WALL	668	666	-7.67673	-11.7222	-4.00366	0	0	-0.01168	-0.01573
WALL	598	663	-21.7192	-13.735	12.231	0	0	-0.03395	-0.02597
SLAB	725	724	3.57174	2.51516	-2.91606	0.006488	0.005431	0	-0.0004

4.4 Plate shear forces

The plate elements for which the maximum and minimum stress resultant values occurred were considered critical and the plate shear forces are listed below:

Table-4 Plate shear forces

TYPE OF ELEMENT	ELEMENT ID	NODE ID	QX	QY
			N/mm	N/mm
SLAB	916	950	-144.792	6.76502
SLAB	912	918	72.4439	-144.961
WALL	666	663	20.7667	27.8954
WALL	629	628	-0.86764	-3.58885
SLAB	920	923	-81.4332	1.46774
SLAB	942	980	-3.26	-9.94873
SLAB	760	790	144.857	14.9253
SLAB	756	758	11.3624	145.026
WALL	664	663	-17.252	20.4386
WALL	627	625	-20.6296	-27.9041
WALL	588	559	2.35948	-2.6433
WALL	668	666	0.864794	3.73441
WALL	598	663	-17.252	-2.84886
SLAB	725	724	-3.20947	9.85917

4.5 Support reactions

The support reactions at in the global directions at the following nodes showed uplift:

Table-5 Support reactions

NODE	FX	FY	FZ	MX	MY	MZ
	N	N	N	N mm	N mm	N mm
1070	0.163066	0.189251	**-21689.8**	21.8628	-18.3963	0
1071	0.010371	0.06758	**-17067.4**	28.2916	9.81309	0
1072	-0.00369	0.042403	**-20571**	28.2295	-4.65306	0
1075	0.007949	0.009007	**-3931.94**	3.94965	-0.174219	0
1076	0.004428	0.006037	**-2546.12**	2.28526	-0.0174042	0
1077	0.005681	0.000454	**-1781.49**	2.05721	0.0463994	0
1078	-0.00017	0.001624	**-931.982**	2.23209	0.0776772	0
1093	-0.01141	0.010856	**-210.149**	1.27142	2.2894	0
1122	0.199813	-0.21416	**-16783.7**	-29.9257	-23.4822	0
1123	0.008053	-0.07871	**-9114.33**	-39.0965	11.8466	0
1124	-0.01134	-0.0488	**-13835.2**	-40.0458	-5.51353	0
1127	0.007695	-0.01481	**-4683.71**	-13.9813	-0.0682868	0
1130	0.016845	-0.00981	**-1254.3**	-7.89211	-0.342997	0

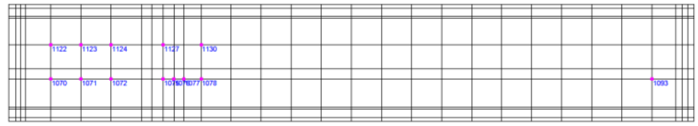


Fig-6 Points where negative vertical support reactions occur on the top slab

5 DEMAND VERSUS CAPACITY RATIOS

Based on the finite element analysis for the static loads the demand versus capacity ratios for the slab and wall elements were calculated for the various modes as follows:

5.1 Slab element

5.1.1 Flexure, [3]

Slab elements were checked for flexural capacity per unit run with the flexural demand developed per unit run and were found to be safe. The ultimate moment of resistance of slab element of overall depth 200 mm was determined. The reinforcement in the slab is 10 mm diameter at 300 mm c/c both ways at top and bottom and effective depth for section is 170 mm. Moment capacities of the concrete slab along both the axes, M_u is 15.50944 KNm/m. This capacity was taken for both slab bottom and top as the reinforcement patterns are the same both ways in diameter and spacing and the same value was compared with the demand. The demand versus capacity ratios were calculated for all the critical elements and are listed below:

Table-6 Flexure demand versus capacity ratios for slab elements

TYPE OF ELEMENT	ELEMENT ID	NODE ID	DEMAND in KN m/m				INTERACTION RATIO			
			Bottom		Top		Bottom		Top	
			Mux	Muy	Mux	Muy	Mux	Muy	Mux	Muy
SLAB	916	950	0	2.332716	-1.63243	0	**0**	**0.15**	**0.10**	**0**
SLAB	912	918	0	1.414575	-2.73776	-0.16085	**0**	**0.09**	**0.17**	**0.01**
SLAB	920	923	0	4.568791	-0.9839	0	**0**	**0.29**	**0.06**	**0**
SLAB	942	980	0	1.829623	-3.23228	-0.67503	**0**	**0.11**	**0.20**	**0.04**
SLAB	760	790	0	2.343034	-1.82908	0	**0**	**0.15**	**0.11**	**0**
SLAB	756	758	0	1.970037	-2.36172	0	**0**	**0.12**	**0.15**	**0**
SLAB	725	724	0	1.815552	-3.21068	-0.67479	**0**	**0.11**	**0.20**	**0.04**

5.1.2 Shear

Area of steel, to resist shear over a length of 1000 mm is 522 mm^2 (as both top and bottom reinforcements are present at all sections). Percentage of reinforcement in the section is 0.26 %. The design shear strength of slab [3] is 75.48 KN. This capacity was taken for both directions as the reinforcement patterns are the same both ways in diameter and spacing and the same value was compared with the demand. The demand versus capacity ratio for all the critical elements is listed below:

Table-7 Shear demand versus capacity ratios for slab elements

TYPE OF ELEMENT	ELEMENT ID	NODE ID	DEMAND in KN/m		DEMAND VERSUS CAPACITY RATIO	
			QX	QY	QX	QY
			N/mm	N/mm	N/mm	N/mm
SLAB	916	950	-144.792	6.76502	1.918283	**0.0896267**
SLAB	912	918	72.4439	-144.961	**0.959776**	1.920522
SLAB	920	923	-81.4332	1.46774	1.078871	**0.0194454**
SLAB	942	980	-3.26	-9.94873	**0.04319**	**0.1318062**
SLAB	760	790	144.857	14.9253	1.919144	**0.1977385**
SLAB	756	758	11.3624	145.026	**0.150535**	1.9213831
SLAB	725	724	-3.20947	9.85917	**0.042521**	**0.1306196**

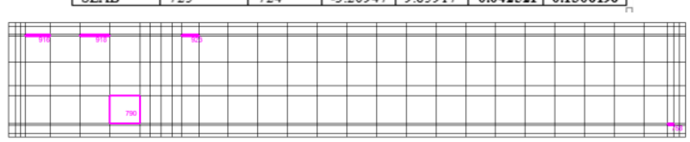


Fig-7 Slab elements showing demand versus capacity ratio>1

The demand versus capacity ratios marked in red are greater than 1 and the corresponding elements are located at the slab wall interface. This may be due to the fact that the chamfered portion in the slab wall junction as seen in Fig-1 (b) is not modeled in the FE model. There is no axial load for shell elements as evident in the table (6), and hence P-M interaction for slab elements is not plotted.

5.2 Wall element

5.2.1 Axial load – Uni-axial moment interaction

Wall elements were checked for axial load and uni-axial moment capacities with the respective demand per unit run of the wall using interaction curve for a typical wall section with the following cross-sectional properties:

Overall depth of the section is 300 mm, longitudinal reinforcement is 12 mm dia @ 150 c/c and transverse reinforcement is 10mm dia @ 200 c/c. Area of steel over a length of 1000 is 754 mm^2, and were found to be safe.

Table-8 P-M Interaction values for wall elements

P_u (KN)	M_u (K Nm)	P_u/f_{ck} b D	$M_u/f_{ck}bD^2$
0.07735	2.5	1.28917E-05	0.001388889
0.10108	0	1.68467E-05	0
0.021068	2.02	3.51133E-06	0.001122222
0.01572	0	0.00000262	0
0.02596	3.37	4.32667E-06	0.001872222
0.1012	1.88	1.68667E-05	0.001044444
0.01564	1.65	2.60667E-06	0.000916667

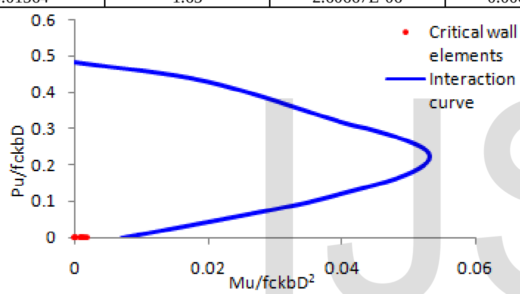


Fig-8 P-M Interaction curve for critical wall elements

5.2.2 Shear, [3]

Area of steel, A_{sv} to resist shear over a length of 1000 mm is 1058 mm^2 (as both top and bottom reinforcements are present at all sections) and percentage of reinforcement in the section is 0.3526 %. Design shear strength is 149.328 KN**.** This capacity was taken for both directions as the reinforcement patterns are the same both ways in diameter and spacing and the same value was compared with the demand. The demand versus capacity ratios for all the critical elements is listed below:

Table-9 Shear demand versus capacity ratio for wall elements

TYPE OF ELEMENT	ELEMENT ID	NODE ID	DEMAND in KN/m		INTERACTION RATIO	
			QX	QY	QX	QY
			N/mm	N/mm	N/mm	N/mm
WALL	666	663	20.7667	27.8954	**0.139068**	**0.186806**
WALL	629	628	-0.86764	-3.58885	**0.00581**	**0.024033**
WALL	664	663	-17.252	20.4386	**0.115531**	**0.136871**
WALL	627	625	-20.6296	-27.9041	**0.13815**	**0.186864**
WALL	588	559	2.35948	-2.6433	**0.015801**	**0.017701**
WALL	668	666	0.864794	3.73441	**0.005791**	**0.025008**
WALL	598	663	-17.252	-2.84886	**0.115531**	**0.019078**

6 MAXIMUM BASE PRESSURE

Area of contact of base raft and the foundation is 56.4 m^2. Total pressure on the soil due to lead wall (in addition to the existing pressure on the soil) considering the whole of contact area between the soil and foundation is 44.7712KN/m^2. This value is less than the SBC of the soil at 2.5m depth of 250 KN/m^2 and hence the foundation base pressures were assured to be within limits.

7 PUNCHING SHEAR CHECK FOR THE BASE SLAB, [3]

Considering 1m run of the footing loaded with lead wall throughout the length of the wall, the check for punching shear is evaluated for 300 thick bottom slab with a reinforcement of 10mm dia @300 mm c/c.

Permissible shear stress in concrete = $k_s\tau_c = \left(0.5 + \frac{b}{D}\right)\tau_c' = \left(0.5 + \frac{b}{D}\right).0.25.\sqrt{f_{ck}} = \left(0.5 + \frac{300}{1000}\right).0.25.\sqrt{20}$ = 0.8944 Mpa

Shear capacity of concrete is 0.8944 times the area resisting two-way shear and it is 429.3250 KN. The total load due to 1 m lead wall is 88.35 KN and self weight of the footing is 29.25 KN. Using a load factor of 1.5 to get the effect of unaccounted loads, upward soil reaction is 117.5 KN/m^2 and the foundation shear is 112.896 KN which is less than shear capacity.

8 FLEXURE CHECK FOR THE BASE SLAB, [3]

Moment developed by the bottom slab of, at the face of the wall, due to loading is 21.15 KN m per meter run of the slab.

Moment of resistance of the section is 61 KN m per meter run of the slab.

Area of steel required to resist the developed moment is 249.58 mm^2.

Available flexural reinforcement in 1 m run of base slab of the footing is 754 mm^2.

9 CONCLUSION

Based on the strength evaluation of the foundation the following conclusions were made on the safety margins available for the foundation for the lead shield loading:

1. The top slab elements are safe for the bending moments developed in them.
2. The wall elements are safe in taking up both the axial load and uni-axial bending moments developed in them and this is clear from the interaction curve.
3. The base pressure developed below the foundation is less than the SBC of the soil assumed at founding level.
4. The bottom slab elements are safe against the flexure and punching shear developed in them due to the loading.
5. The top slab elements show higher demand versus capacity ratio for shear forces developed in them (>1) at the slab wall junction loaded with the lead wall.
6. Uplift forces are developed in the interior points of the top slab
7. Maximum vertical displacement of the nodes as per the present analysis is less than 1 mm.

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