

DESIGN OF THE FOUNDATION SLAB FOR A DRY FUEL STORAGE BUILDING IN THE UK

Steve Bennett

Civil Design Engineer, Sir Robert M^cAlpine Ltd., UK

ABSTRACT

This paper describes the design of a reinforced concrete Foundation Slab that supports casks used for the interim storage of spent fuel. The slab is seismically qualified and designed as a Nuclear Safety Related Category 1 structure to ACI 349. The slab dimensions are 114 m x 59 m x 0.75 m with capacity for 146 casks. It supports a steelwork superstructure that is not Safety Related.

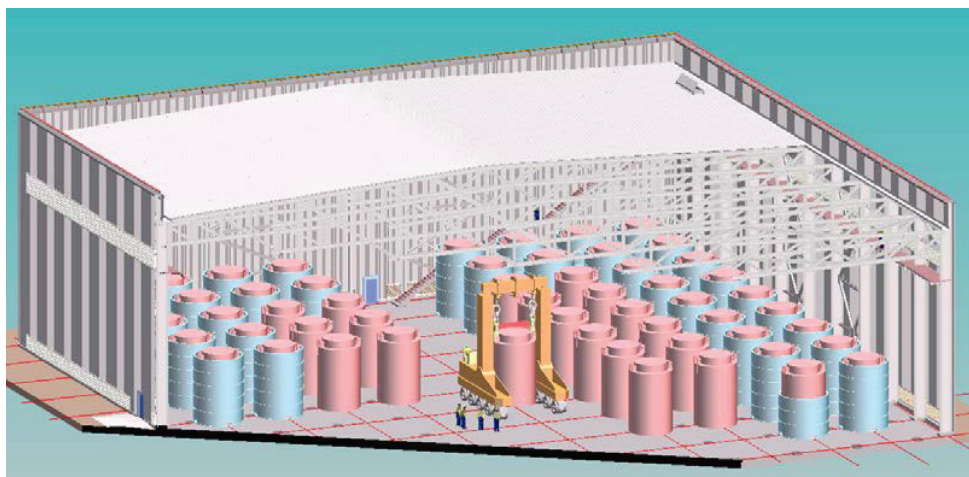
Design challenges:

- Each 3.6 m diameter cask weighs 250 t. The casks are stored on a 5.7 m x 5.4 m grid; the loading sequence is not fixed, so any pattern of cask loading may occur.
- Temperature. The design temperature for the cask / slab interface is 100°C. This is a normal load. The maximum predicted compressive force for load combinations including thermal is 15400 kN / m run. The maximum tensile forces are resisted by 2 layers of 32 mm diameter reinforcement in each face.
- 100 year design life with a crack width limit of 0.3 mm.
- Potential liquefaction / seismic settlement required the slab to span locally with a maximum slope of 5° to ensure cask stability.

The basic slab design included a response spectrum FE analysis undertaken using MSC.NASTRAN.

Thermal analysis was undertaken using DIANA in a non-linear analysis. Both tension and compression limits were approached and so assumptions about restraint were invoked.

For temperatures >66°C ACI requires special consideration of concrete properties. Testing of the designed concrete mix determined the effect of elevated temperature, which was shown to be a 10% reduction of 28 day strength.



INTRODUCTION

Extensions to the operating life of Nuclear Power Stations lead to the generation of increased quantities of spent fuel. In some cases this exceeds the design capacity of the original spent fuel storage system, e.g. ponds. For some stations the strategy is to construct a Dry Spent Fuel storage facility, which will eventually accommodate all the Spent Fuel up to and beyond the end of station life. For the facility described in this paper, the spent fuel is transferred into a Dry Storage canister and moved into the Dry Fuel Storage Building (DFSB) where it is stored in free-standing casks.

The dry fuel storage casks are 3.6 m diameter cylinders 6 m high. They are double skinned steel filled with concrete and have an internal air flow with the fuel cooled by natural convection. Ambient air is drawn in at the base of the cask and hot air exhausts from the top. The casks are arranged on the slab in a 5.7 m x 5.4 m grid, but provision is made in the design for that arrangement to be changed, and for a cask to be placed in any location on the slab. The building will be gradually filled with storage casks to suit power generation schedules and end of life timescales, this is expected to take approximately 50 years.

The casks are designed to resist loads that arise due to seismic effects; however, they are not designed to impact each other. Therefore the principal design requirement for the slab is to support the casks and prevent them from colliding with each other.

Shielding rings are an additional provision to increase shielding. They take the form of steel and concrete rings that are lifted over the cask to form an external shield. They significantly increase the load on the slab.

The Foundation Slab dimensions are 114 m x 59 m x 0.75 m thick. The plan dimensions allow for the storage of 143 No. casks, and are limited by site constraints and economy. The thickness is the same as used for similar slabs in the USA (30") and the design demonstrates that this thickness is sufficient to resist the applied loads. The slab is seismically qualified.

Figure 1 shows a plan view on the slab when the building is full of casks.

The slab supports a clad steelwork envelope with a single clear span; its purpose is to satisfy planning requirements. The envelope has low and high level louvres permitting flow of air in at ground level that is exhausted at high level, i.e. the building is passively ventilated by natural convection. The interior is not environmentally controlled. Therefore, at various times through its life the slab may be subject to both low ambient temperatures and high internal temperatures caused by the cooling casks.

The superstructure is not seismically qualified as the casks are capable of resisting any impact loads generated by structural collapse.

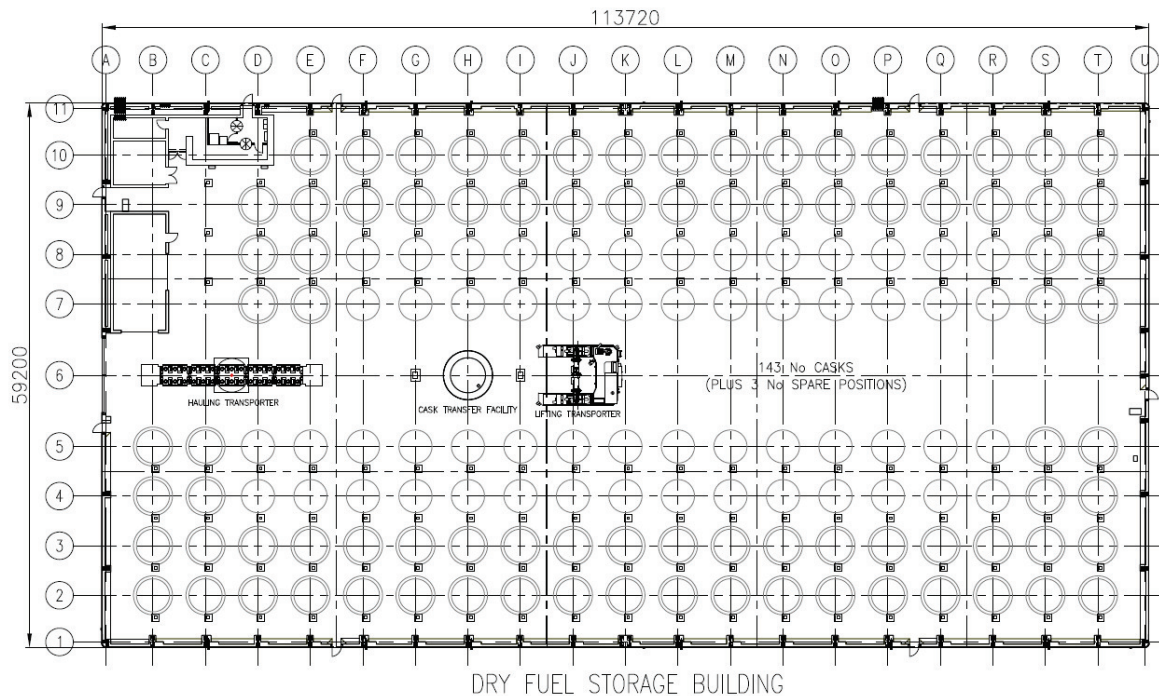


Figure 1. Slab general arrangement

As well as normal design loads with a return frequency of 1 in 100 years, the slab must offer sufficient support to the casks when subjected to Hazard loading, where the load represents a 1 in 10,000 year event. The main load in this category is the Safe Shutdown Earthquake, with a site specific zero period acceleration (zpa) of $0.14g$ for the 10^{-4} hazard peak horizontal ground motion.

All elements of the structure are designed for a 100 year design life with appropriate detailing to provide a maintenance-free structure as far as possible.

All of the design and analysis described in this paper was undertaken by Sir Robert McAlpine Ltd. Nuclear Design Division.

DESIGN CODES AND STANDARDS

The Foundation Slab is a Nuclear Safety Related Category 1 structure designed in accordance with the requirements of ACI 349. This is consistent with the original design principles of existing structures on the site. As British contractors and suppliers are not familiar with working to American Codes the construction specification was written to include the most onerous requirements of either US or British codes with the acceptance criteria expressed in terms of British Standards.

MECHANICAL LOADING

The most significant applied loads are those arising from the cask dead weights. Each cask weighs 250 t when loaded with spent fuel. At the expected spacing that represents an unfactored distributed load of 81 kN/m^2 . With shielding rings in place, this increases to 105 kN/m^2 .

The superstructure reactions are applied to the Foundation Slab to determine structural design requirements. For global design, the effects of superstructure reactions for the normal operational load combinations are obtained from static analyses. The effects of reactions for SSE are directly produced from the combined dynamic analysis model, and OSE reactions are scaled from the SSE reactions.

For slab design local to the superstructure columns, loads are obtained from the superstructure analyses, including the design basis hazard wind and snow loads at 1:10,000 year return period.

THERMAL LOADING

A CFD analysis was performed by others to determine the temperature regime within the building. The casks have an internal chimney and the fuel is cooled by natural convection. Cool air is drawn in at the base and hot air is exhausted from the top of the cask. As the building envelope is not sealed the internal space may be subject to the same temperature variations as the exterior. At ground level the slab is generally subject to external temperatures which for design purposes range from +29.5°C to -13°C.

The base of the casks is heated by the fuel stored within, up to a maximum design temperature of 100°C. The base of the cask is directly in contact with the Foundation Slab so the design temperature is applied directly to the floor slab.

ANALYSIS

Structural Analysis

A dynamic soil-structure interaction (SSI) analysis of the Foundation Slab and steelwork superstructure was performed, following the Impedance Method of ASCE 4-98, to determine the stresses and forces in the Foundation Slab for a range of cask loading conditions. The structure was modelled and analysed using the MSC NASTRAN FE code with Lower Bound, Best Estimate and Upper Bound ground conditions modelled using frequency-independent soil stiffness and damping values (soil compliances) for six degrees of freedom for the selected loading conditions.

The main load in this condition is the safe shutdown earthquake (SSE) consisting of PML UK soft site spectrum normalised to 0.14g horizontal zpa. A suite of artificial time histories matching this input spectrum were used in a time history analysis, applied to the base of the slab simultaneously in the three orthogonal directions. The Response Spectrum Analysis method was used to define the response of the structure in terms of displacements, forces, moments and stresses, which were combined with results from a static analysis. The slab reinforcement was designed for the combined results to ensure that the structure remains essentially elastic.

The most significant forces arise at the interface between the loaded and unloaded areas of the slab. Because of the unknown loading sequence the interface must be assumed to be feasible at any location, and therefore the design reinforcement is provided throughout the entire slab.

Figure 2 shows the displaced shape of the slab when fully loaded at the east end. There is a central aisle for vehicle movements and so two areas of the slab are depressed by the cask loading. Figure 3 shows maximum displacements under three different cask pattern load conditions.

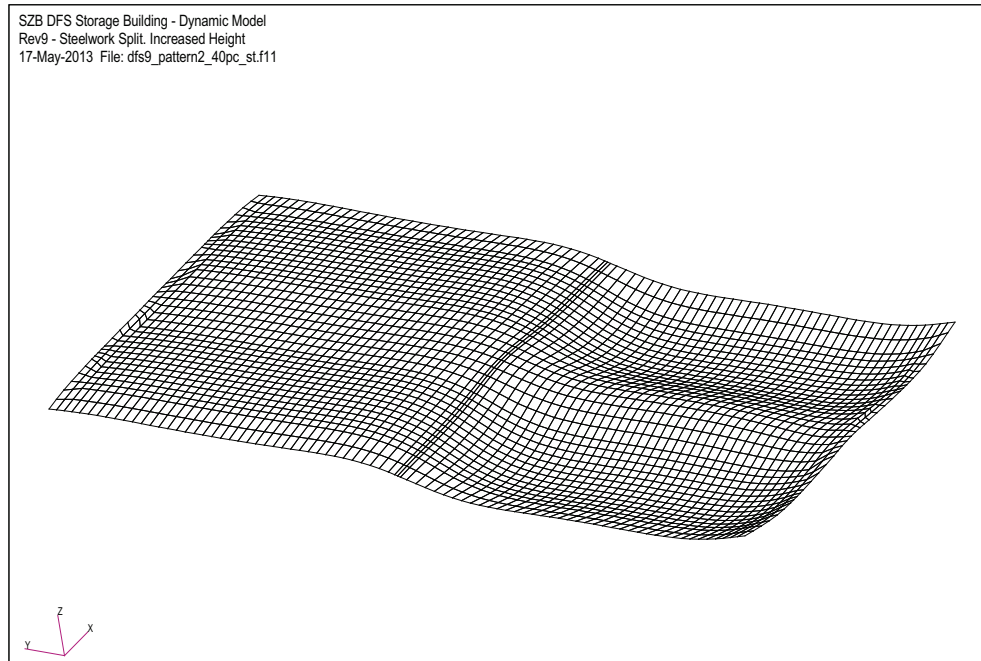


Figure 2. Slab displaced shape, static loading.

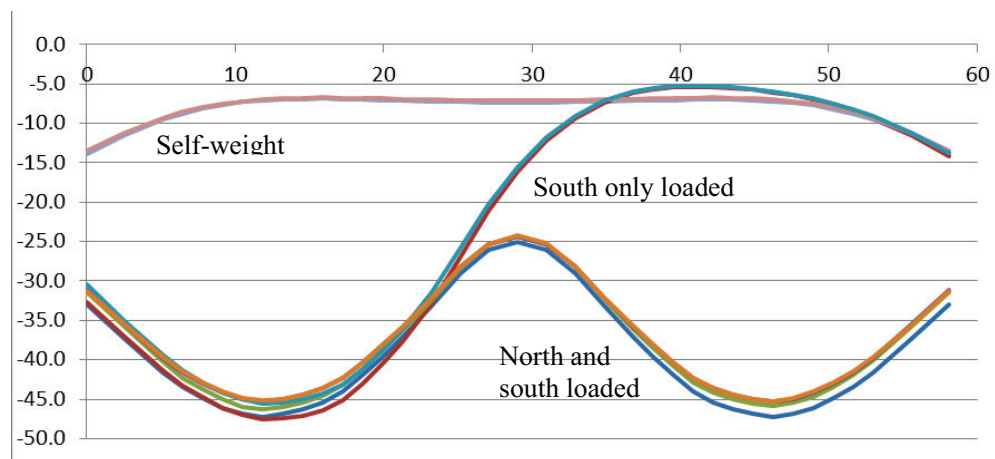


Figure 3. Maximum displacements across width of slab.

Thermal Analysis

DIANA FE was used to create 2D flat plate models of a representative part of the slab to determine how temperature loads affect the unloaded areas of the slab. A group of casks was modelled and the maximum in-plane stresses arising used to determine reinforcement requirements, see Figure 4.

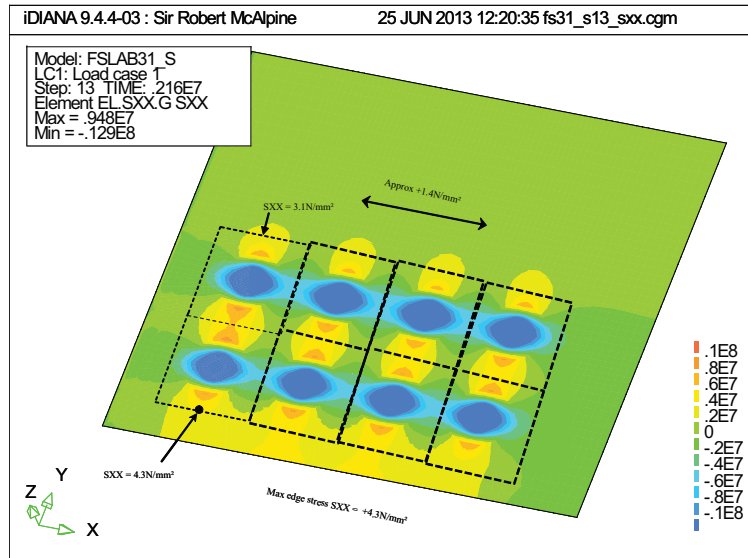


Figure 4. Slab thermal ‘global’ effects.

The area local to a single cask was analysed using a detailed 3D model of a quarter cask and associated slab to determine how local temperatures affect stresses in the concrete and reinforcement, see Figure 5.

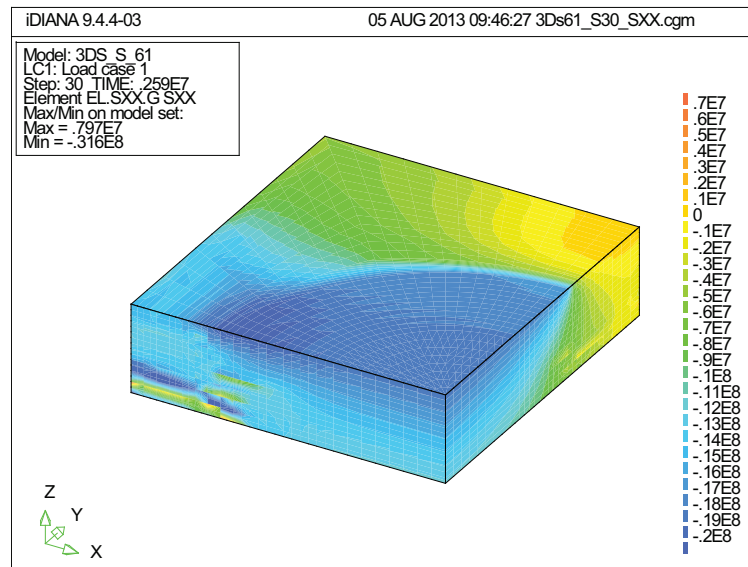


Figure 5. Slab thermal local effects.

The boundaries were varied between fixed and free. The material model included variations between elastic and non-linear smeared cracking. The reinforcement was explicitly modelled in these analyses and reinforcement stresses extracted directly from the FE analysis results.

When the boundaries were fully fixed, the compression caused by the thermal loading exceeded the capacity of the slab. That case is considered too conservative, as local expansion will cause the slab to expand globally. A slip membrane at the underside of the slab is included in the construction

requirements facilitating this movement. With the analysis boundaries allowed to move to replicate the anticipated slab expansion the maximum predicted compressive force for normal load combinations including thermal is 15400 kN/m run compared to a conservative code strength of 17000 kN/m run.

CASK TIP-OVER AND SLIDING

A non-linear rigid body motion analysis was undertaken to determine the stability of the cask when supported by a sloping slab and subject to the SSE input. The system was analysed considering a level slab and also a slab with an arbitrary slope of 5°. Coefficients of friction between the cask and the slab of 0.2 and 0.8 were assessed.

The cask was modelled using ANSYS with mass distributed as in the real cask and allowed to separate from the floor. Due to the circular shape of the base of the cask it can roll if the applied force is sufficient for it to lift up on an edge. Input motion was defined using Secondary Response Spectra derived from the structural analysis and applied as a displacement time history.

Figure 6 shows the mesh for the cask and supporting section of slab, and a results plot for the flat slab with high coefficient of friction. In this scenario, the cask lifts an edge and then rotates on the edge of its base, translating across the slab. The plot shows movement of a point on the top of the cask.

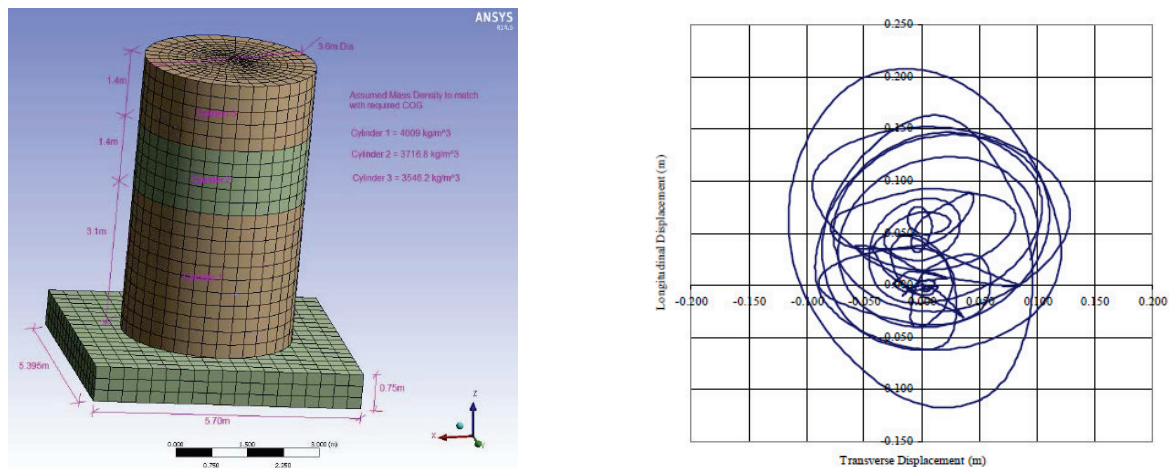


Figure 6. Model of cask for tip-over analysis and displacement plot.

The minimum Factor of Safety (FoS) against overturning is 10. The casks are shown to slide when the coefficient of friction is low. A FoS for sliding and rocking was calculated as half the clearance between casks divided by the maximum radial displacement of the top of the cask. The minimum value for this FoS is 2.2.

In a design-basis seismic event, the casks must not tip over or move to such an extent that collisions between casks could occur. The analysis concluded that the casks would potentially move, but not sufficiently to collide with each other.

Therefore a 5° slope is acceptable with respect to cask tip-over and sliding and is a design criterion for the main slab, particularly when considering potential settlement in the event of local liquefaction.

POTENTIAL LIQUEFACTION AND RESIDUAL SETTLEMENT

A geotechnical investigation identified that there is limited potential for some liquefaction to occur in a band of soil at some depth below the slab. It was also identified that there is the possibility of some residual seismic settlement, with a recommended maximum design value of 150 mm. The precise areas of potential settlement are not known and so the design strategy was to accommodate any loss of support due to residual settlement from small pockets through to large areas. This is treated as a design basis hazard load.

Three scenarios are considered:

- a small pocket of settlement which the slab is capable of spanning.
- a larger area of settlement which the slab is not capable of spanning.
- the effect of settlement at the perimeter of the slab where a cantilever occurs.

The 5° slope shown to be acceptable for cask rocking and sliding in combination with the maximum residual settlement of 150 mm was used to define an acceptable slab geometry. The slab must be able to span

$$2 \times 0.150 / \tan 5^\circ = 3.4 \text{ m} \quad (1)$$

assuming the formation of plastic hinges and a 3 point failure. The slab is designed to span an unsupported 3.4 m as a minimum. If it spans further, then the slope after failure is less steep and the geometry is acceptable. The edge of the slab is considered as a cantilever subject to the same geometrical constraint. For the edge of the slab there is no direct cask load but the loads applied by the steelwork superstructure are considered.

The edge of slab cantilever is shown to be sufficiently robust so as not to collapse in a condition where the slope of the floor exceeds 5°. That 5° slope has been demonstrated to not compromise the integrity of the casks. This assessment considers the reduced capacity and any effect on the design margins. For the residual settlement case a cantilever at the edge of the slab must be able to span 1.715 m to keep the slope at <5°. The nearest cask to an edge is at 5.56 m; therefore cask instability is not an issue.

DESIGN

Reinforcement design for the results of the dynamic analysis was undertaken manually with the components of moment and force combined in accordance with the principles of Wood and Armer using in-house developed spreadsheets. The results were enveloped and rationalised to define the reinforcement requirements.

The slab design considered the load combinations and factors presented in ACI 349 with the individual load case results and reinforcement requirements extracted from the analyses presented in the preceding sections.

DESIGN REINFORCEMENT

The reinforcement is specified as grade 500C for increased ductility.

Design reinforcement is 2No. layers of C32 bars at 200 mm centres in each face in each direction. In the 750 mm thick slab this equates to 383 kg/m³. Although this is a high density the regular nature of the structure means that once a fixing methodology is determined the reinforcement fixing is relatively straightforward. Shear links of 20 mm diameter are provided at 400 x 200 mm centres throughout the slab. A typical view of fixed reinforcement is shown in Figure 7.

For Normal Operating combinations the maximum utilisation is in the top of the middle of the slab in the North-South direction. For Design Basis Hazard Loading combinations the maximum utilisation is parallel to the edge of the slab.



Figure 7. Slab reinforcement

To ensure a ductile response the design strength for shear is lower than that for flexure, and shear reinforcement is provided throughout the slab. Reinforcement detailing is in accordance with the general requirements of Chapter 12 of ACI 349.

CONCRETE

Ideally the design and construction would use a consistent set of codes throughout. However, British Contractors and suppliers are not familiar with working to American Codes. Therefore the specification was written to include the most onerous requirements of either US or British codes with the acceptance criteria expressed in terms of British Standards, e.g. cube testing.

The concrete is specified as grade C40/50. To mitigate against early thermal cracking the mix was designed to reduce heat of hydration by the use of 35% PFA replacement and limestone aggregate. The use of limestone aggregate had the effect of lowering the achievable strength. To achieve the required strength the concrete mix has a high cement content. Trials showed that 440 kg/m³ of cementitious content was necessary.

For temperatures >66°C ACI 349 requires special consideration of concrete properties. Trials were undertaken to determine the effect of the 100°C design temperature on the concrete strength. Cubes were heated and crushed at 100°C. The results showed that a strength reduction of up to 10% could be expected compared to control cubes tested at ambient temperature. The mix was designed to achieve a characteristic 28 day strength which when reduced by 10% exceeded the required design concrete strength.

The PFA content caused the initial strength gain to be slower than for concrete containing CEM I cement alone, however, even taking into account the reduction for thermal effects the concrete achieved the required 28 day characteristic strength. The significant mechanical and thermal loading is imposed on the

slab after a period of years. As the slab is supported by the ground it is reasonable to consider the concrete strength at a time later than 28 days to be representative of its practical design strength. The concrete had adequate 28 day strength and showed a further strength increase of 15% between 28 and 56 days. This gives confidence that structural performance is adequate with good margins.

A high slump was preferred for pumping and placing of the concrete in the heavily reinforced slab. Water reducing admixture was specified to enhance workability and facilitate satisfactory compaction of the concrete.

Durability

The Foundation Slab is designed to satisfy the 100 year design life and account for the likely operating regime inside the building (e.g. elevated temperatures).

The slab is designed to limit crack widths under serviceability conditions to 0.3mm, applicable for normal durability requirements as CIRIA Report C660.

To provide a structure with the maximum probability of remaining serviceable during a 100-year design life the cover to reinforcement was selected to exceed the minimum requirements of the design codes. For internal surfaces 50 mm cover is specified and for outer and bottom surfaces of the slab 75 mm cover is specified.

CONCLUSION

The design of the slab was required to meet many objectives, one of these was to use the thickness of a typical Foundation Slab used elsewhere and design the reinforcement to fit that slab. In addition, objectives of robustness to resist the applied mechanical, seismic and thermal loads, durability for the 100 year design life, allowance for potential residual settlement and code compliance had to be demonstrated. These objectives were achieved.

Several different, independent and complex design scenarios had to be considered and the results combined in a realistic way to determine the final reinforcement requirements. Although each component of the design is addressed using standard techniques the combination of results generated stress values which were at or close to the design limits allowed in both tension and compression.

REFERENCES

- American Concrete Institute (2006), ACI 349M-06, *Code Requirements for Nuclear Safety-Related Concrete Structures*, US
- American Society of Civil Engineers (1999), ASCE 4-98, *Seismic Analysis of Safety-Related Nuclear Structures*, US
- Bamforth P. (2007), CIRIA Report C660, *Early-age Thermal Crack Control in Concrete*, London
- Ishiyama J (1982), "Motion of Rigid Bodies and Criterion for Overturning by Earthquake excitations", *Earthquake Eng and Structural Dynamics*, Vol 10, 635-650.
- Wood, R.H., "The Reinforcement of Slabs in Accordance with a Pre-Determined Field of Moments," *Concrete*, V. 2, No. 2, 1968, pp. 69-76. (discussion by Armer)