Transactions, **SMiRT-26**
Berlin/Potsdam, Germany, July 10-15, 2022
Division V

ITERATIVE STATIC SOIL STRUCTURE INTERACTION ANALYSIS FOR UK HPR1000

Barnali Ghosh[1], Vipul Kumar[2], Rob Talby[3], Ming Tan[4] Zhang Tao[5]

[1,3,4] Technical Director, Mott MacDonald, UK (barnali.ghosh@mottmac.com)

[2] Senior Engineer, Mott MacDonald, UK (vipul.kumar @mottmac.com)

[5] Principal Engineer, Civil Engineering Department, China Nuclear Power Engineering Co.Ltd (zt_csn@cgnpc.com.cn)

ABSTRACT

Soil-Structure interaction plays a vital role in understanding the behaviour of massive structure such as nuclear power plants founded on compliant soil during static and dynamic loading conditions. Using the work undertaken for the Generic Design Assessment in the UK for UK HPR1000 [GNSL, 2020], this paper focus on presenting the methodology outlined below to evaluate the Static SSI. The impedance method is generally not suitable for UK sites, which are generally much softer and may exhibit significant non-linear soil response. This paper proposes an innovative technique to model the soil behaviour within the context of the impedance method using iterations between the structural and geotechnical models to achieve convergence. It will be shown that better representation of the layer beneath the foundation makes it possible to have more accurate value for the settlement and the forces and moments on the raft (in addition these values are more conservative).

INTRODUCTION

The UK HPR1000 is a Pressurised Water Reactor using the Chinese Hualong technology with electric output of approximately 1180MW. The UK HPR1000 has evolved from a sequence of reactors that have been constructed and operated in China since the late 80s, including the M310 design used at Daya Bay and Ling'ao (Units 1 and 2), the CPR1000, the CPR1000+ and the more recent ACPR1000. The first two units of CGN's HPR1000, Fangchenggang NPP Units 3 and 4, are under construction in China. Fangchenggang NPP Unit 3 is the reference plant for the UK HPR1000.

With the intention to be deployed to the Bradwell 'B' site in the UK, the UK HPR1000 was put forward for GDA in January 2017, to be assessed jointly by the regulators - Office for Nuclear Regulation (ONR) and the Environment Agency. The regulators provided independent scrutiny to ensure that the reactor design is applicable to UK regulatory standards of safety, security and environmental protection. The GDA for the UK HPR1000 was successfully completed in February 2022, with the issuing of a Design Acceptance Confirmation (DAC) from the ONR and a Statement of Design Acceptability (SoDA) from the Environment Agency [ONR, 2022].

Soil-Structure interaction (SSI) plays a vital role in understanding the behaviour of massive structure such as nuclear power plants founded on compliant soil during static and dynamic loading conditions. Using the work undertaken for the Generic Design Assessment in the UK for UK HPR1000 [GNSL, 2020], this paper focus on presenting the methodology outlined below to evaluate the SSI. Foundation-soil interaction is a function of relative foundation-soil stiffness ratio and is influenced by Soil stiffness, Foundation stiffness and Foundation size (thickness and plan dimensions). Most nuclear codes consider the Impedance Function Method and the Direct Method for evaluation of Static Soil Structure Interaction. The impedance method is generally not suitable for UK sites, which are generally much softer and may exhibit significant non-linear soil response. Traditionally for a large-scale nuclear power plant, the Common Raft Foundation supports the Reactor Building and other important buildings. Usually, Soil Structure Interaction for the static case is performed on a global model which covers the Nuclear Island by determining appropriate ground parameters and creation of a flexibility matrix which is used in local structural models. In this paper it will be shown that better representation of the Soil Structure interaction between the raft and the founding soil can be achieved by following an iterative process to account for the rigidity of the Common Raft Foundation, to derive the settlement springs for the structural assessments.

DEVELOPMENT OF GENERIC GROUND PROPERTIES

At this generic design stage, the only site information available to characterize the ground is the assumed shear wave velocity (V_s). Various international codes such as the European Utility Requirements for LWR Nuclear Power Plants (EUR), the Canadian Foundation Engineering Manual, the International Building Code (IBC) and AASHTO LRFD Bridge Design Specification contain generic recommendations for various ground properties based on generic ground classification groups – Soft, Medium and Hard (i.e. the ground types).

Table 1 summarises the generic soil properties that are often used for GDA process. Additionally, another site has been classified as Very Soft which has a shear wave velocity of 150m/s for the UKHPR1000. These properties have been specified in the EUR (European utility requirements for LWR nuclear power plants, Volume 2, Chapter 4, Rev D, Oct 2012) for different ground conditions which are defined based on shear wave velocity. It is however noted that these properties can be used for seismic analysis only and refer to undrained soil behaviour. All the SSI results presented in this paper are for very soft site where settlement is often a critical element while designing raft foundation.

Table 1: Soil Properties from European Utility Requirements for LWR Nuclear Power Plants

Type of Spectrum	Very Soft	Soft			Medium			Hard		
Shear wave velocity(m/s)	150	250	350	500	600	800	1100	1200	1700	2500
Mass Density (kg/m^3)	2000	2000			2200			2500		
Poisson's Ratio	0.49	0.47			0.4			0.35		
Free Field Shear Modulus G_{max} (MPa)	45	125	245	500	792	1408	2662	3600	7225	15625

GEOTECHNICAL SETTLEMENT ANALYSIS

This section provides preliminary settlement calculations to derive equivalent static ground springs for very soft sites (shear wave velocity =150m/s). These springs are further used for structural design works for the Generic Design Assessment (GDA) application. The preliminary settlement calculations have been carried out using analytical methods, using geometry and loads of the proposed buildings. When a soil is loaded by a building it will deform due to:

1. Deformation of soil grains.

2. Compression of air and water contained within the soil void space; and
3. Squeezing out of water and air from the void space.

Points (1) and (2) relate to immediate distortion settlement and (3) relates to consolidation. Consolidation behaviour of soils is typically modelled by oedometer testing. This section will outline the methodology followed to calculate the total settlement for typical ground profile for the very soft site using generic site properties.

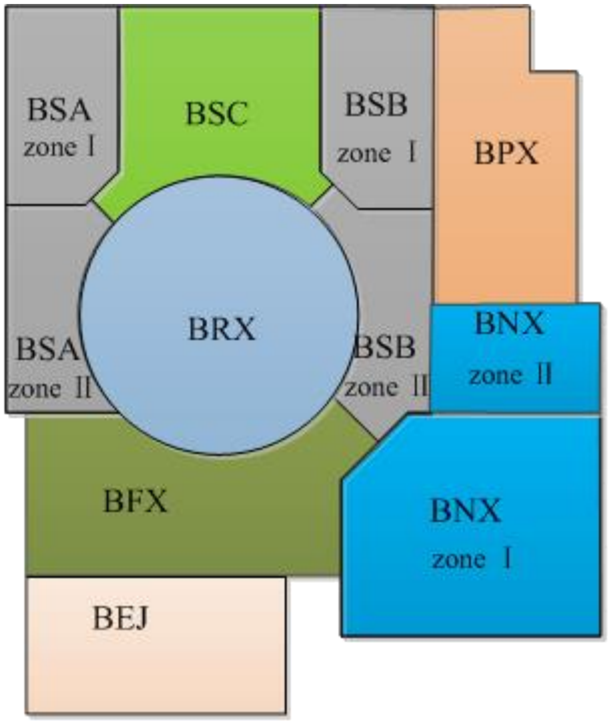

Figure 1: Generic Site Layout of main structures of UK HPR1000

The general layout of main structures for UK HPR 1000 is shown in Figure 1. The Common Raft Foundation is a 110 m long by 82 m wide reinforced concrete raft foundation. The base level of the Common Raft Foundation is -11.800 m. It is assumed that the construction sequence for these buildings involve the following steps

1. Dewatering of the groundwater (2 months)
2. Earthworks to main excavation level (12 months)
3. Construction of foundations, buildings and loading (30 months)
4. Installation of equipment (24 months)

The service life for the plant is assumed as 60 years and design life is 100 years to include construction/ decommissioning. Settlement predictions were made for 100 years.

GDA Ground Model

For typical consolidation settlement analysis, it is necessary to define a ground model. For generic site

- Superficial Deposits to 3.7m depth (between levels 6 to 2mOD), which are above the foundation levels, so the stiffness of this layer is not important for settlement consideration.
- London Clay Formation 43m thick (between levels 2 to -41mOD)

- Deep deposits of Lambeth Group 42m thick above Chalk (between levels -41 to -83mOD)

The coefficients of volume compressibility mv of Lambeth Group, London Clay Formation and superficial deposits are shown in Table *2*. These values are selected based on published information for these materials. Ground stiffness can be considered in terms of axial stiffness modulus in long-term condition E', which is correlated to mv for isotropic condition as (e.g. Carter and Bentley, 1991):

$$E' = \frac{(1+\nu)(1-2\nu)}{m_v(1-\nu)}$$

where ν is the drained stiffness which is taken equal to 0.2. The values of over consolidation ratio (OCR) have been inferred from the graph by Duncan and Buchigani (1976) based on characteristic undrained stiffness characteristic undrained shear strength values shown in Table *2*. The ground parameters are not based on any site-specific data but instead they reflect the publicly available information for over consolidated London clay soil and standard parameter correlations.

Table 2: Proposed Lumped parameters used in the settlement analysis for each layer

Geological strata	Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Undrained Elastic Modulus (MPa)	Drained Elastic Modulus (MPa)	m_v (m^2/MN)	Recompression consolidation index c_r
Superficial Deposit	18	20 – 40	5 – 10	5	0.2	0.2
London Clay	19	50 – 250	40 – 100	32 – 80	0.04 – 0.018	0.035
Lambeth Group	20	250 - 800	100 – 250	80 - 190	0.015 – 0.007	0.03

Site specific settlements have been derived using two methods

- Method 1 – using Oasis Pdisp software and long-term drained elastic modulus (E').
- Method 2 – using Rocscience's Settle3D software programme and recompression consolidation index, C_r. Applied loading less than pre-consolidation pressure assuming OCR>2.

Both methods assume the foundation is 'fully flexible', and there is no account for soil-structure interaction (i.e. amendment of the settlement due to structural stiffness). The average settlement from these two methods has been used to derive springs. The site-specific settlements from the two methods above are based on net applied pressures at the underside of the foundation. The net applied pressure is taken as

$$Net\ \ pressure = Gross\ \ pressure - ground\ water\ uplift - unload\ from\ excavation$$

The site-specific springs can be calculated as

$$Site\ specific\ springs = \frac{Net\ Load}{Total\ settlement\ \ at\ centroid\ \ of\ individual\ buildings}$$

As the Common Raft Foundation is located at the base of an excavation at approximately 12m depth, there is a significant difference between the gross and net bearing pressures. The structural analysis will require gross pressures; however, the long-term soil settlement will be dependent on net bearing pressures (based on the changes in effective stress). In general, the ratio between gross and net pressure is around 1.5 (i.e.

the total net load is approximately 60% of the total gross load) but varies across the common raft from building to building as the load is redistributed between the different iterations.

Method 1

Software Pdisp (19.3) by Oasys uses different input parameters than Settle3D. Pdisp does not consider stages (dewatering, excavation, loading) and for this reason net pressure (applied structural pressure less overburden pressure due to excavation and less buoyancy pressure) is considered for the analysis. The characteristic drained elastic modulus stiffness (E') provided in Table *2* is used as input to Pdisp. The stiffness parameters are provided for different geological formations and levels. These values correspond to large strains.Figure 2 presents the predicted settlement from this method. The maximum settlement at the centre of the BRX building is about 350mm.

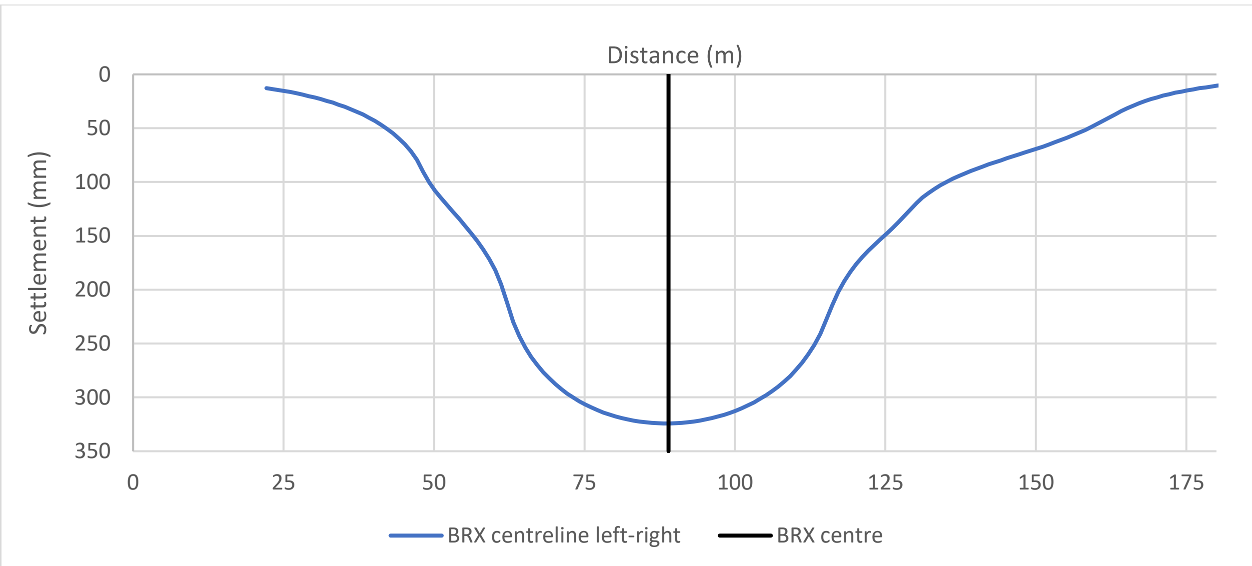

Figure 2: Predicted PDISP settlement due to net load along centreline of BRX

Method 2

Software Settle3D (4.0) by Rocscience was used for accessing settlement in this method. This program uses specific ground parameters for non-linear settlement calculations. A three-dimensional view of the foundation model is shown in Figure *3*. The following sequences are considered in the analysis

- A sequential construction of the various buildings where it was assumed that construction of foundations, buildings and loading was completed at 30 months.
- Net loading was applied at the underside of the foundation or slab.
- Ground water was assumed at the ground surface for the analysis.
- Settlement was predicted at the end of the design life.

The settlement analyses performed for this GDA work was undertaken by applying the Net Pressures at the formation level in the consolidation analysis. Net Pressures for each building were obtained by deducting the total stress due to the excavation, also known as the total overburden (which is the effective stress due to the excavation plus the porewater pressure) from the Gross Pressure of each building.

During the initial excavation works, swelling (heave) will be induced in the short term. However, since the unloading/reloading of the soil is considered as elastic, any short-term heave will cease when a fraction of Gross Pressure of a building becomes equal to the reduction in the total stress due to the excavation. This would be the point when the soil stress becomes similar to its in-situ value. Thereafter, due to the application of the remaining fraction of Gross Pressure (or, the Net Pressure) the soil experiences net positive loading which will result in long-term settlement.

From the construction viewpoint, heave will be induced in the short-term during excavation at which point it will be removed prior to the foundation being placed. The soils below foundation will experience a net positive loading and hence long-term settlement, which is what has been assumed in our analyses.

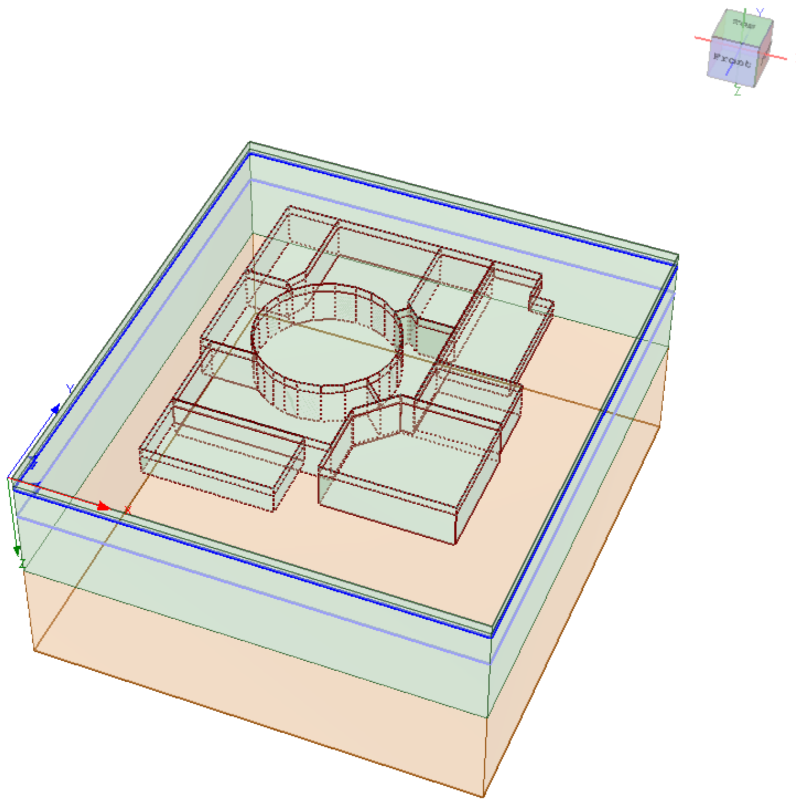

Figure 3: Three-dimensional view of the calculation model by Settle3D with ground layering

ITERATIVE SSI ANALYSI

Vertical ground springs subject to static loadings should ideally be derived from site specific settlement analysis using the proposed loads, geometry, geology and ground parameters, construction sequence etc for the site. The settlement is used derive the vertical ground springs for the first preliminary iteration. The settlements from the above method are based on net applied pressures at the underside of the Raft Foundation. These spring values are provided as the input in the global finite element model of the Raft Foundation. The settlement predicted are based on a 'fully flexible' foundation with no account for 'rigidity' for the stiffness of the Common Raft Foundation thickness. These settlement predictions are considered to be preliminary estimations and are used as the first iteration with no account of soil-structure interaction. Global finite element models with structures supported on rafts are created with the ground response modelled using a series of springs.

The ground springs typically have a uniform stiffness equivalent to the soil's modulus of sub-grade reaction. Figure 4 illustrates the behaviour of the raft foundation on springs compared to the soil continuum. If a raft of uniform stiffness is subject to uniform loading and is analysed using soil 'springs', then it will settle a uniform amount. However, this does not always reflect the true behaviour of the raft as the raft of finite stiffness will settle more in the centre than at the edges. This is especially true for the due the size of raft.

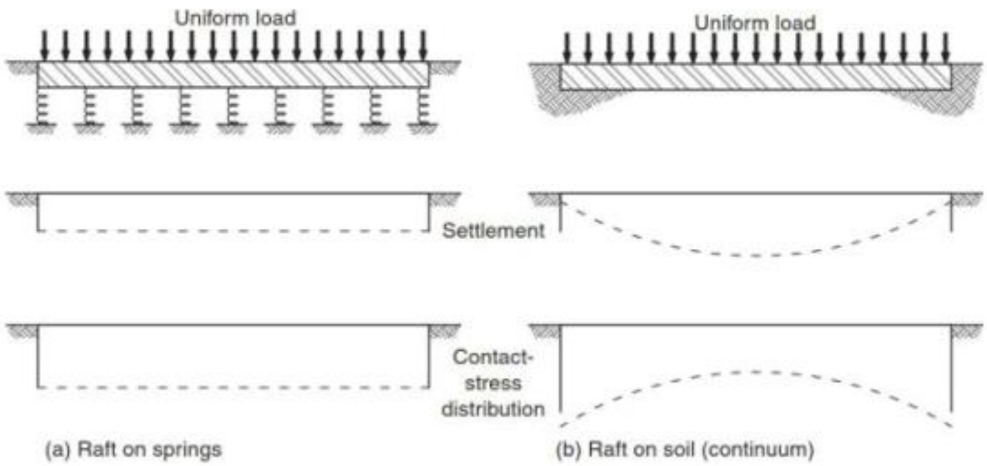

Figure 4: Raft on Spring compared to Raft on soil (MOGE 2012)

To account for the rigidity of the Common Raft Foundation, a series of iterations has been performed between the geotechnical model for the common raft and the structural model to derive the settlement springs for the structural assessments. Figure 5 presents the iteration workflow adopted to derive the springs and differential settlement for the buildings in the common raft

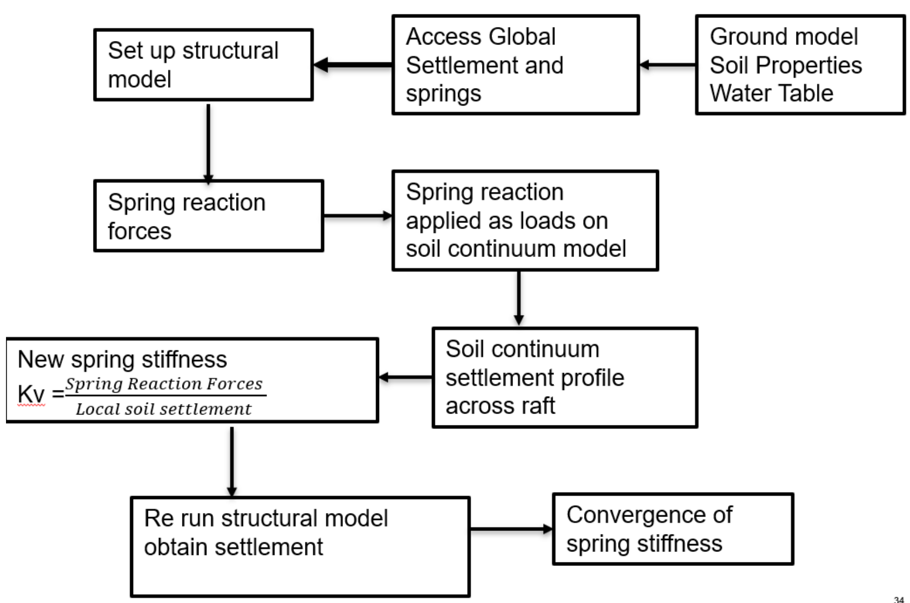

Figure 5: Iteration Workflow for Static SSI

In the first iteration BRX was the heavier than BSB-II. Further on we have adopted the MOGE-type approach for the iteration of pressures and springs for the Common Raft between the uncoupled structural model and the geotechnical model. The structural model included the full rigidity of the common raft and superstructure and the pressures distribution in each successive iteration reflect that behaviour. In further iterations BSB-II became the most heavily loaded building with gross pressure equal to 694kPa. Figure 6 below shows the gross pressures for all the iterations. The dotted lines represent the average pressure for all the buildings across the common raft. It can be seen that with each successive iteration, the average pressures are converging across the foundation, reaching an asymptotic value with the average pressures converging to a mean value of about 580kPa.

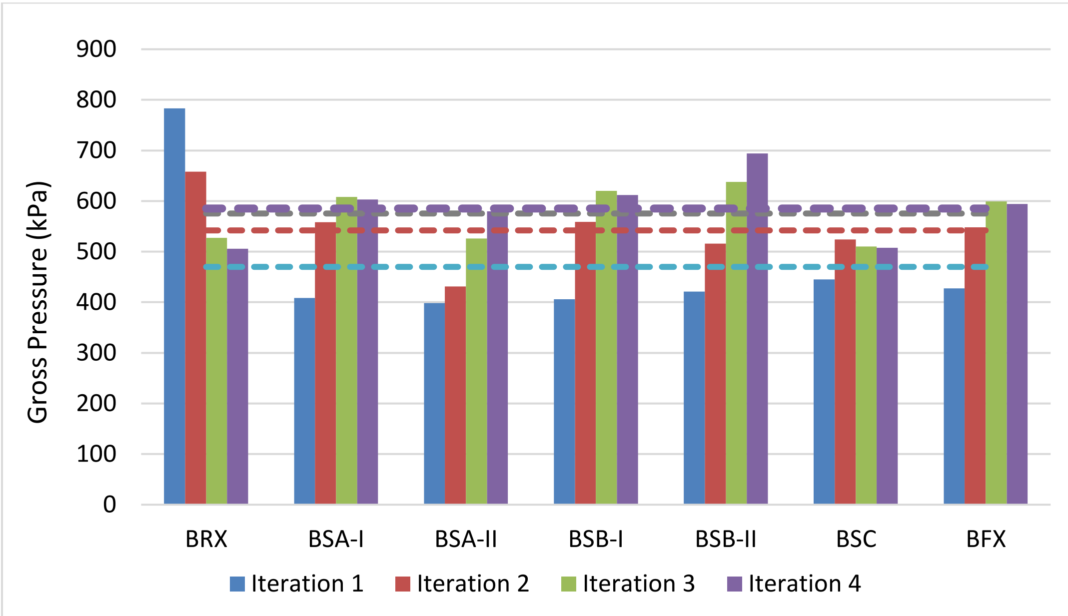

Figure 6: Change in Gross Pressures for each successive iteration

Both Pdisp and Settle3D methods assume the foundation is 'fully flexible', and there is no account for soil-structure interaction (i.e. amendment of the settlement due to structural stiffness). The average settlement from these two methods has been used to derive the vertical ground springs. The settlements from the two methods above are based on net applied pressures at the underside of the foundation. It can be seen that the springs are converging from each successive iteration. Figure 7 shows that the weighted average change in spring stiffness values between each iteration based for the buildings on the common raft foundation. It can be seen that % change between iterations 2 and 3 is 3.8% and 3.2% between iteration 3 and 4. Both of these iterations are below the 5% convergence criteria recommended in the Manual of Geotechnical Engineering-2012 and it can be concluded that convergence has reached.

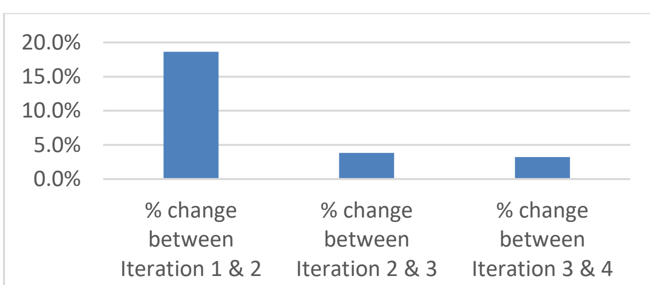

Figure 7: Average Variation in spring value for successive iteration

DISCUSSION & CONCLUSION

Settlement predictions are conservative from the geotechnical models as they are modelling the flexible foundation behaviour. We have attempted to calculate the differential settlement using the settlement profile from the structural model for one of the buildings. This is an approximate calculation and just shows the mechanism. Adjusting the settlements from the structural model for buoyancy and excavation effects (the ratio of net to gross pressure is about 0.6 across all iterations) the settlement from the structural model matches very well the settlement from the flexible geotechnical model. This also proves that convergence has been achieved after the iterations. After the application of the credible solution, the tilt for BNX-II can be fixed at 0.1 degree and the differential settlement can be calculated as 150mm, which is the upper bound. This value can be controlled by proper construction phasing and other optioneering arrangements for the piping structure where it is considered critical.

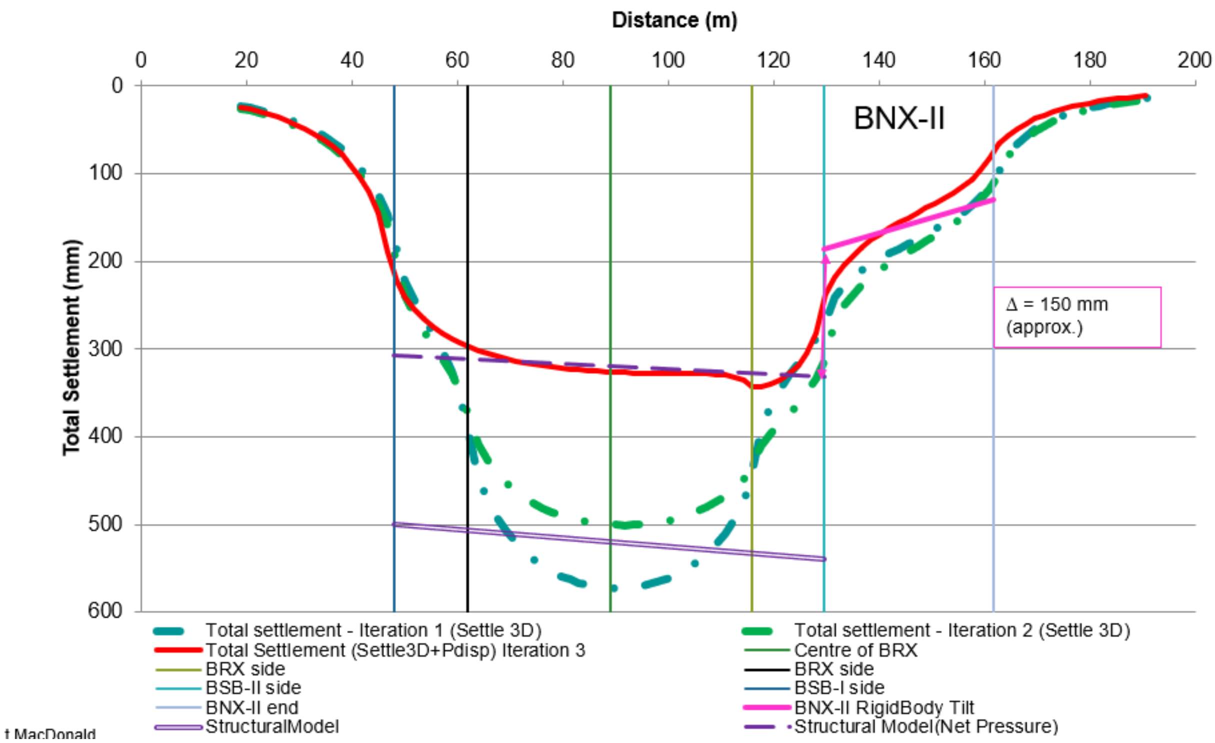

Figure 8: Calculation of differential settlement from Structural model of Common Raft

It should be noted that the actual behaviour of the foundation loaded area will vary between flexible and rigid assumptions due to SSI. The settlement during construction vs consolidation phases will be different and the construction sequence needs to be modelled in detailed design phases. However, at the detailed design phases the following aspects should be evaluated in greater detail

1. Use of constant soil stiffness - In the site-specific study the soil layers will be assumed to be horizontal. The stiffness of the major consolidation layer will be varied during the consolidation process. The unweathered London Clay will be divided into sub layers to allow for the varying stiffness value with depth. The in-situ drained Young's Modulus, E_0' will be normalised against the in-situ mean effective stress, $p_0{}'$ for the settlement calculations.
2. The use of uniform spring stiffnesses under respective buildings without regard for edge effects – In the site-specific analysis stage, springs should be derived based on the actual unequal pressure distribution in the building which will ensure that springs are varied across the building base to

account for edge effects. In most cases it will ensure that softer springs are in the centre and stiffer springs are at the edges of the building.

3. Not considering any proportion of the imposed load when evaluating long-term settlements - In the site-specific calculations a proportion of the live loads (usually 30%) should be added to the dead load for settlement calculations. This is in line with the recommendations of most nuclear codes and their load combinations.
4. Not considering the effects of creep and construction staging when considering the stiffness of the concrete within the SSI analysis. – In the site-specific work, the effects of construction staging, and creep will be included in the model by using long term stiffness modulus for concrete.

This Paper also shows an innovative technique to model the soil behaviour within the context of the impedance method using iterations between the structural and geotechnical models to achieve convergence. It is shown that better representation of the layer beneath the foundation makes it possible to have more accurate value for the settlement and the forces and moments on the raft (in addition these values are more conservative).

REFERENCES

[1] European Utility Requirements for LWR Nuclear Power Plants, Volume 2, Chapter 4, Table 4, 2012
[2] Carter, M., Bentley, S.P. (1991) Correlations of soil properties. Pentech Press Publishers: London, pp. +60
[3] ICE Manual of Geotechnical Engineering (MOGE), 2012, www.icemanuals.com, Chapter 56, Rafts and Piled Rafts.
[4] GNSL (2020), *Pre-Construction Safety Report Chapter, ,HPR/GDA/PCSR/0016, Rev 001.*
[5] American Society of Civil Engineering (2017), *Seismic Analysis of Safety-Related Nuclear Structures, ASCE/SEI 4-16.*
[6] Settle3D, Rocscience, User Manual, (https://www.rocscience.com/software/settle3)

[7] Eurocode 7: Geotechnical design – Part 1: General rules BS EN 1997-1:2004+A1:2013
[8] Duncan, J.M., Buchigani, A.L. (1976) An engineering manual for settlement studies. Geotech. Eng. Report, Dept. of Civil Eng., University of California at Berkeley
[9] Horikoshi, K., Randolph, M.F. (1997) On the definition of raft-soil stiffness ratio for rectangular rafts. Geotechnique, 47(5), 1055-1061.
[10] Skempton, A.W. (1951) The bearing capacity of clays. Building Research Congress, England
[11] Berezantsev, V.G., Khristoforov, V.S, Golubkov, V.N. (1961) Load bearing capacity and deformation of piled foundations. Proceedings of the 5th International Conference on Soil Mechamics and Foundation Engineering, Paris, vol. 2, pages 11-15
[12] Stroud, M.A. (1988) The standard penetration test – its application and prediction. In: Penetration testing in the UK, Proceedings of the geotechnology conference organized by the Institution of Civil Engineers, Birmingham, pp 29-49
[13] MIL-HDBK-1077/3 (1997) Soil Dynamics and Special Design Aspects. US Department of Defence