Highbank power station – concrete-encased turbine and generator plinth replacement design process

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ASBTRACT

Manawa Energy, a renewable energy provider in New Zealand, operates a network of 26 power schemes from the Bay of Plenty in the North to Otago in the South. The Highbank power station stands as one of Manawa's highest strategic value assets, featuring an existing 26.5MW generator and turbine scroll case assembly. Undertaking the full replacement of these components, particularly the concrete-encased turbine, presents a unique challenge not previously attempted in New Zealand. The total volume of concrete is approximately 310 m^3, with a maximum diameter of 10 m and a height of 7 m. Collaborating closely with Manawa Energy, WSP has been engaged in a transformative project aimed to bring a new life to the Highbank power station, which has been in operation for approximately 80 years.

The key challenges that the design engineers faced included understanding the static and dynamic actions including soil-structure interaction, the interaction of the existing station superstructure with the mass concrete encasement, and the design of mass concrete to transfer seismic and operational loads. The design engineers utilized a 3-dimensional strut-and-tie model to quantify actions in the mass concrete, along with a combination of force and displacement-based analysis to ensure compatibility between the various components.

This paper describes the analysis, design, and methodologies deployed to address the challenges for the replacement of the concrete-encased turbine and generator plinth.

1 INTRODUCTION

Manawa Energy Ltd is Aotearoa New Zealand's largest independent electricity generator and renewables developer. They currently operate 26 power generation schemes across New Zealand from the Bay of Plenty in the North, down to Otago in the South. This paper focuses on the structural engineering aspects of Canterbury's Highbank power station redevelopment, which stands as one of Manawa's highest strategic value assets. The station features a single generator combined with a vertical axis Francis turbine assembly (the Unit). The new unit is more efficient, and can achieve a higher rated output with the same hydraulic conditions compared to the existing unit.

Collaborating closely with Manawa Energy and the wider design team, WSP has been engaged in this transformative project aimed to bring a new life to Highbank power station by improving the efficiency and resilience of the station for at least another 40 years of operation.

1.1 Background

The Highbank power station is the downstream end of the Rangitata Diversion Race (RDR) which was the first major river diversion project in New Zealand. The RDR comprises a 66 km long diversion canal that starts at the Rangitata River in Canterbury and flows northwards across the Canterbury Plains with an average fall of 1 ft/mile (~190 mm/km) to the Rakaia River. At the northern end of the canal, there is a 100.5 m drop down the Rakaia River terraces into the Highbank power station.

The canal and power station were constructed throughout World War II, this led to challenges with staffing, funding, and provision of suitable materials, particularly high-quality steel for the power station equipment which was challenging to source for non-war associated works.

The Rangitata River has a high coarse sediment load which causes severe wear of the turbine seals and associated equipment. In the 1970’s the station generation capacity had reduced to approximately 65% of the original rated capacity due to sediment build-up and wear on mechanical components. This led to the turbine runner being replaced along with improvements in sediment control at the canal intake and ongoing removal of sediments to reinstate the original performance.

1.2 Existing Structural Form

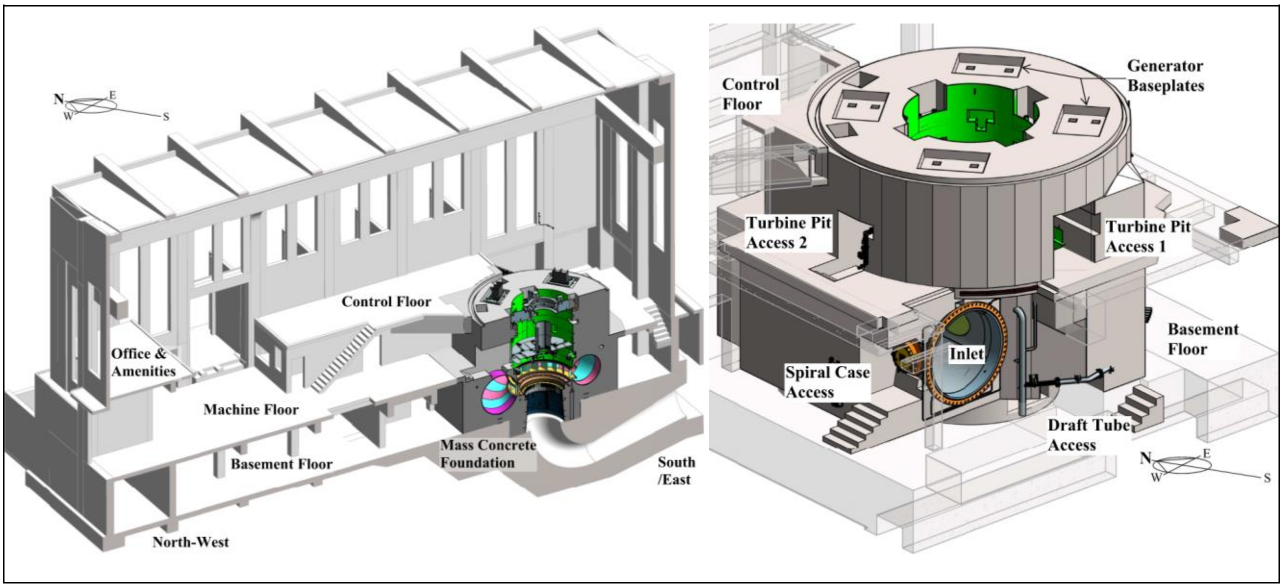
The Highbank power station is a reinforced concrete building with a large machine floor with a 90-tonne gantry crane, two levels of ancillary offices and amenities, a control floor adjacent to the unit, and a basement under the full building footprint with workshop and storage facilities. The roof includes a shallow-pitched suspended concrete slab supported by reinforced concrete moment frames. Later works added a lightweight sloped roof over the existing. The foundations comprise a concrete raft slab with beam thickenings around the perimeter. The suspended reinforced concrete slab on the ground floor is supported internally by beams and columns, and externally by perimeter concrete walls.

The station is relatively compact being 36.5 m long, 15.8 m wide and 15.8 m high above external ground level. The basement level is approximately 3.7 m below ground with the foundations under the unit extending a further 5.5 m below basement level locally to allow for the draft tube bend.

The centreline of the main inlet valve (MIV) and turbine horizontal plane aligns with the half-height between the basement and ground floor level. The bottom of the generator is raised 3.0 m above the ground floor level with the area between formed with circular concrete walls tied into the floors at each level. The generator upper bracket supports all vertical and horizontal loads from the rotor, shaft and runner and is supported by a steel stator frame which acts as a two-way moment frame fixed to the top of the concrete.

1.3 Project Extent

Today, the Highbank power station has been operating for nearly 80 years and the operating equipment is considered to be at its end of life. Manawa Energy has investigated a range of options to maximise the value of the available water. It concluded that the most efficient option is to replace the entire generator and turbine between the MIV and the top of the draft tube, along with all ancillary mechanical and electrical equipment to improve the efficiency, minimise silt impact, and provide reliable generation for the next 40 years. Figure 1 illustrates two building views that show the various components interacting directly with the concrete, where new concrete is shown in dark grey, existing concrete is light grey and embedded steelwork is shown in various colours based on the type and function from the manufacturer.



(a) Longitudinal section through centre of Unit *(b) Enlarged View of Unit*

Figure 1: Isometric Station Views

The condition of the existing spiral case was investigated during the feasibility stage to confirm whether it could be re-used. However, the high wear due to sediment load at the station, combined with the poor-quality steel used for the existing spiral case and associated embedded components require replacement to meet the project objectives. This presents a unique challenge that has not previously been attempted in New Zealand on a machine of this size. In most cases, efficiency improvements are made by replacing mechanical components and resurfacing and or re-finishing the wearing surfaces of cast-in components.

Developing the concept scope for the project included identifying the minimum extent of concrete that required replacement. This confirmed that demolition of all existing concrete around the scroll case from the basement level up including the concrete above which supports the Generator and adjacent suspended floor was required. The total volume of concrete to be replaced is approximately 310 m^3. This extent is concentrated in a small area with a maximum diameter of 10 m and a height of 7 m.

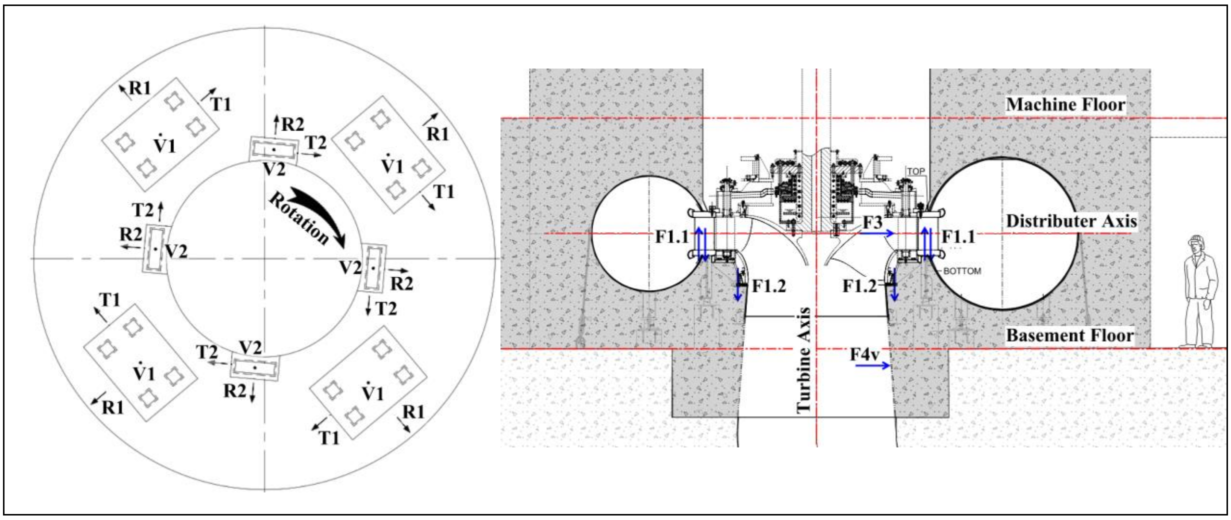
In this paper, the design loads are described, and the design challenges and methodologies to resolve them are presented. These challenges include the interaction of the existing structure and new mass concrete, soil-structure interaction, developing an analytical model of the mass concrete to consider the effects of concentrated loads, fatigue, shrinkage, and constructability.

2 DESIGN LOADS

The mechanical loads applied to the concrete by the new generator and turbine systems are substantial. A thorough understanding of these loads, specifically those acting concurrently with seismic forces, allowed the WSP design team to formulate multiple load combinations for application in our analytical models. The development of these design cases involved collaboration with the mechanical consultant, Regenerate NZ Ltd, and the equipment suppliers GE Power New Zealand Ltd.

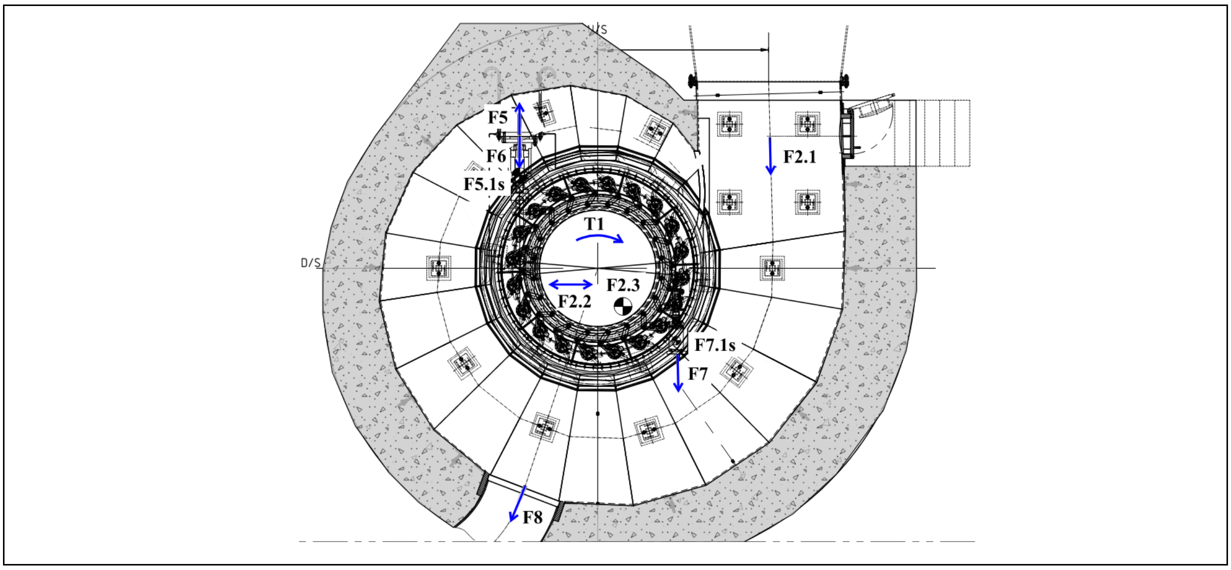
The schematic shown in Figure 2(a) illustrates the reactions from the generator baseplates, where forces R1, T1, and V1 represent radial, torsional, and vertical forces associated with the main generator baseplates respectively. R2, T2, and V2 represent radial, torsional, and vertical forces associated with the generator lower bracket connections. Table 1 indicates load magnitude at each of these locations.

Figure 2(b) and Figure 2(c) illustrate the force locations corresponding to load patterns F1.1, through to F8. Among these load patterns is the reaction force from the large servomotor, which provides a braking mechanism for the turbine. The maximum load comes from the servomotor which is capable of exerting a reaction force of 2800 kN on the surrounding concrete.



(a) Generator Baseplate Mechanical Reactions

(b) Turbine Cross Section Mechanical Reactions



(c) Distributer Axis Plan View, showing mechanical reaction locations

Figure 2: Mechanical reaction loads on the mass concrete.

Table 1: Generator reaction loads

Load ID	Max. load per baseplate / Connection (kN)	Associated Load Pattern
V1	690	Max. Hydraulic Thrust
T1	2837	Out of Phase Synchronisation Torque
R1	590	Nominal (system start/stop), Short Circuit Torque, and ½ Poles Short Circuit (warm)
V2	270	Maintenance (Weight of Rotor, Shaft, and Runner)
T2	10	Braking
R2	301	½ Poles Short Circuit Torque (warm)

Table 2 provides an overview of the load combinations, encompassing forces from the turbine and generator equipment along with seismic forces. The mechanical supplier provided a summary of load patterns deemed reasonable for simultaneous consideration with seismic forces. These load combinations were systematically incorporated into the analytical model to understand and quantify force distribution throughout the concrete encasement and into the supporting elements allowing the design team to provide suitable reinforcing steel to meet the varied demands.

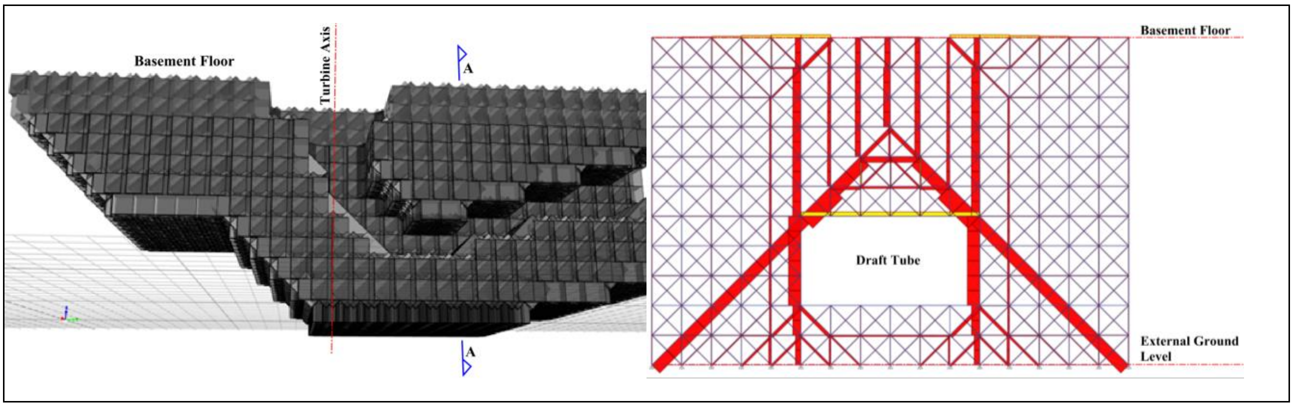
Table 2: Turbine Reactions

Load Case	Design Limit State	Seismic / Non-seismic	Load Description
1	ULS	Seismic	Normal operation, with generator short circuit torque
2	ULS	Seismic	Turbine Maximum hydraulic thrust load (load rejection speed), with generator short circuit torque
3	ULS	Seismic	Turbine Maximum hydraulic thrust load (load rejection speed)
4	SLS	Non-seismic	Normal operation with generator short circuit torque
5	SLS	Non-seismic	Normal operation with servomotors jamming
6	SLS	Seismic	Normal operation maintenance loads
7	SLS	Non-seismic	Normal operation with generator out-of-phase synchronization torque

3 ASSESSMENT OF EXISTING FOUNDATION

The existing foundations of the Station encompass two distinct structural systems. In the South-East, a mass concrete foundation supports the Unit and the surrounding primary structure. This foundation encapsulates the draft tube and extends to a depth of up to 5.5 m below the basement level bearing directly onto the existing dense sandy river gravels. Conversely, the foundations on the North-West side of the Station comprise a concrete raft slab featuring beam thickenings around the perimeter immediately below the basement level floor as shown in Figure 1.

Foundation replacement could be very challenging and impose additional time, and cost into the project and embodied carbon to the environment. The existing mass concrete foundation beneath the main unit is approximately 950 m^3. To replace this with a new reinforced concrete foundation would result in approximately 630 Tonnes of CO^2 equivalent in materials alone (assuming 30 MPa concrete with 75 kg/m^3 reinforcing). A combination of hand calculations and 3D strut-and-tie models, following NZS 3101:2006 and Appendix C5F of NZSEE guidelines, was employed to evaluate the structural performance of the existing mass concrete foundation under the imposed loads from the newly integrated turbine and generator system. The existing structural drawings show a sparse distribution of reinforcing, predominantly situated at floor level and encircling the draft tube. To optimise the utilisation of existing ties, tension limits were applied based on laboratory-tested reinforcing steel properties, and the redistribution of seismic loads was included. Through these measures, the design team were able to validate a load path with adequate capacity within the existing foundation that could save considerable project time and cost and reduce embodied carbon. Figure 3 illustrates a 3D strut and tie model as well as a section cut through the same model. The section illustrates the distribution of force around the draft tube, and through the foundation.



(a) ETABS 3D foundation strut and tie model

(b) Section A-A through foundation model, illustrating load path

Figure 3: ETABS Foundation strut and tie model

4 SUPERSTRUCTURE, MASS CONCRETE AND SOIL-STRUCTURE INTERACTION

Although the concrete mass around the Unit and its associated mechanical elements constitute only 20% of the total station mass, 50% of the total seismic force is transferred through this concrete to the foundations according to the modal analysis results of the full station model. The high stiffness of the mass concrete results in a significant portion of the superstructure's seismic force being transferred to the encasement concrete around the turbine through the ground floor. There was concern about seismically isolating the new concrete from the existing superstructure by disconnecting the suspended floor since the concrete floor slab restrains the concrete mass under SLS loading and assists with minimizing vibrations from the turbine and generator.

The design team employed Soil-Structure Interaction and Displacement-Based Design to achieve a more realistic understanding of the interaction and seismic demands on the concrete mass. To account for soil-structure interaction and ground variability, two full analytical models were developed to determine the seismic force transmitted from the Station superstructure to the Unit encasement concrete: (1) a fixed base model, and (2) line springs under the foundation grillage beams, with area springs assigned to the mass concrete raft foundation.

In the fixed base model, the base shear force at the bottom of the mass concrete was determined to be 24,600 kN, indicating a seismic force transfer of 16,850 kN from the superstructure. The transfer of this seismic force through the concrete mass posed challenges for both the design of the mass concrete encasement and the assessment of the existing foundation.

A lower bound vertical spring stiffness of 10,000kN/m was used to estimate the station foundation's base flexibility which significantly reduced the shear force in the mass concrete encasement for the load case along the building (from 18,000 kN to 8400 kN) and had a lesser impact on earthquakes across the building (from 24,600 kN to 20,000 kN). The base flexibility also led to a decrease in seismic force transfer from the superstructure to the encasement concrete of 800 kN along the building and 12,200 kN across the building respectively. Insufficient data was available to quantify the local soil stiffness variability under the station, this was considered acceptable as minimal changes to the global seismic load path were proposed.

The maximum force that can be transferred into the mass concrete foundation around the draft tube is constrained by the restoring moment from the self-weight of the encasement concrete over, and the flexural resistance from the foundation concrete breakout, which is less than the strength of the reinforcement starter bars into the foundation.

When the hold-down capacity at the base of the encasement concrete is reached, the entire concrete block will experience rocking, potentially occurring during a strong ULS seismic event. According to the Displacement-Based Design (DBD) approach, a minimum resistance of 35,600 kNm at the base is required to limit the concrete mass rocking to a 2.5% drift. The factored uplift capacity of the concrete mass due to its self-weight is 43,150 kNm, meeting the minimum uplift resistance requirements of the DBD approach.

The maximum transferred force from the superstructure to the encasement concrete is 6800 kN based on the overstrength uplift capacity of the block. If the seismic force exceeds this limit, the concrete block will rock, and additional seismic load will be transferred to the perimeter concrete walls of the superstructure. To ensure the prevention of brittle failure before concrete rocking occurs, the design of the concrete mass reinforcement included the additional seismic load in the 3D strut-and-tie model and analysis.

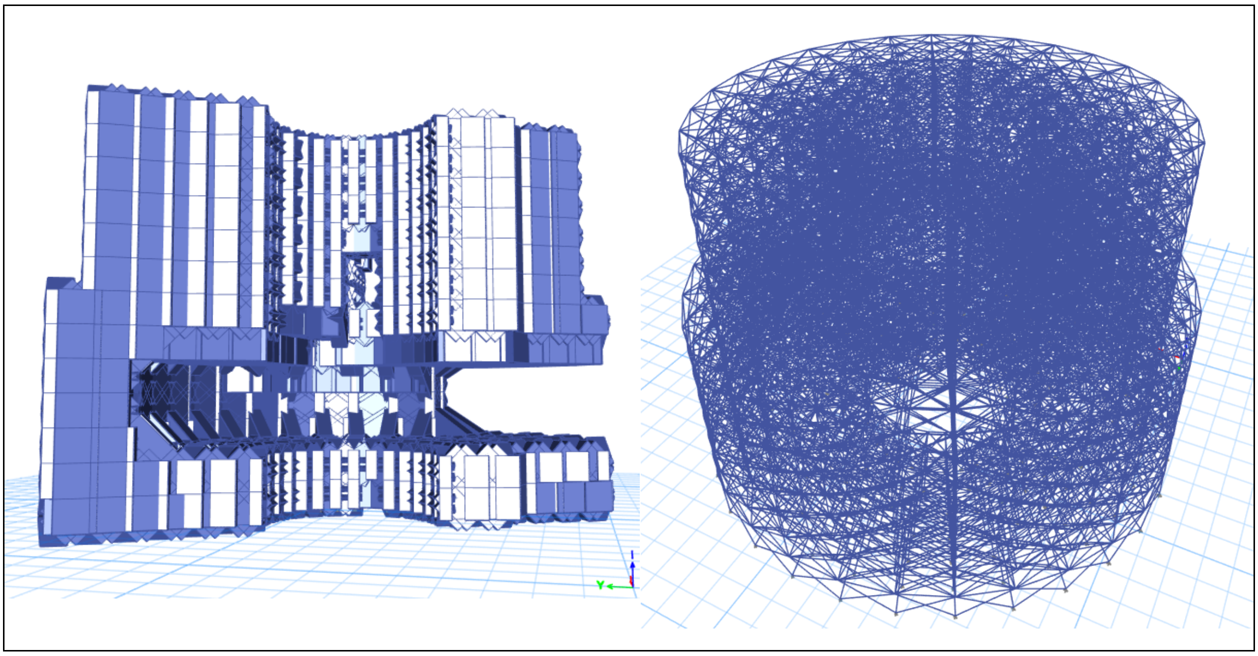
In summary, considering the interaction between the superstructure and mass concrete, 50% of the total seismic force is transferred through the new encasement concrete into the existing mass concrete foundation. Soil-structure interaction analysis showed a 20% reduction in force transferred to the mass concrete. The displacement-based design was able to verify that the concrete block's overall stability could be achieved through the rocking mechanism. Additionally, the maximum force that could be transferred to the concrete block was calculated and used in the design to prevent any brittle failure mechanisms forming.

5 VIBRATION

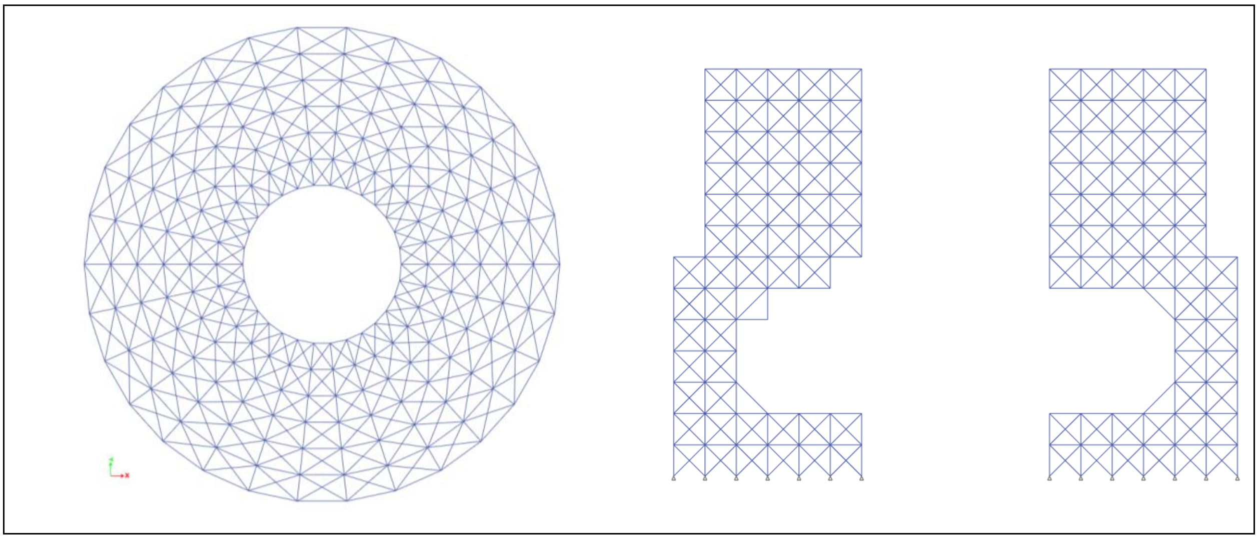
Vibration resonance in the concrete is linked to the excitation frequencies of the turbine, generator, and associated components, compared with the mass, and stiffness of the concrete unit and surrounding connected elements. The excitation frequencies of the new components are similar to the existing unit’s frequencies being Swirl = 1.7 Hz, rotating parts = 5 Hz, wicket gates = 65 Hz, and the runner = 100 Hz. The shape of the concrete mass, connections to the floors, and foundation are being reinstated as near as possible to match the existing. Therefore, the performance of the encasement concrete and machine floor is expected to be similar to the existing structure in terms of vibration, and it has been confirmed that the current arrangement has no vibration issues. Hence vibration analysis of the building and associated elements surrounding the concrete encasement was not considered further.

6 DESIGN OF REINFORCED CONCRETE FOR APPLIED MECHANICAL AND SEISMIC LOADS

The complex load paths and concentrated forces were the primary drivers for developing a 3D strut-and-tie model to analyse the new encasement concrete and mass concrete foundations, rather than relying on more approximate hand calculation methods. The analytical model comprises approximately 17,000 frame elements as illustrated in Figure 4, this was created in ETABS and validated through hand calculations. For each concentrated point load, the load path from the analytical model was followed through to the supporting structure and/or external ground. This process was supplemented with hand calculations to verify the values, ensuring that the complexity of the analytical model and the numerous elements did not lead to any mistakes in the load path.



(a) Extruded Section *(b) Full 3D model*



(c) Plan at the base *(d) Section*

Figure 4: New encasement concrete 3D strut and tie model views

Strut and tie elements were defined following NZS 3101:2006 and Appendix C5F of the NZSEE guidelines. In the conventional 2-dimensional strut-and-tie models, each internal joint is connected to adjacent joints by 8 frame elements (4 strut and ties and 4 struts only). However, in the 3D model, the connection of each internal joint to adjacent joints is provided by 18 frame elements (6 struts and ties and 12 struts only).

Hand strut and tie calculations were developed for locations of substantial mechanical and seismic loading. Bespoke steel detailing was required in these areas to ensure the effective redistribution of loads into the surrounding concrete. The numerical values determined from these calculations were contrasted against the same locations in the global strut and tie model. This process allowed the WSP design team to validate the load paths and ensure that local and global reinforcing was distributed appropriately.

7 FATIGUE AND SHRINKAGE

Fatigue design for reinforcing steel to NZS3101:2006 is simplified such that the reinforcing stress caused by repetitive loading at SLS shall be equal to or less than the range of 50-150 MPa, depending on the bar diameter. This approach does not take account of the actual number of cycles or range of stresses that may occur over the design life. The number of cycles from the servomotor and generator is a key factor for fatigue design, the designers followed ACI 215R-74, "Considerations for Design of Concrete Structures Subjected to Fatigue Loading," as it provides more comprehensive design criteria that correlates the stress limit to the number of cycles, as shown in Figure 5.

In consultation with the Mechanical consultant, the critical fatigue loads of 1400 kN and 700 kN could conservatively be applied up to twice per day from the servomotor and generator, respectively, this results in up to 58,400 cycles in the concrete design life (80 years). The normal operational loads are significantly less than this value and do not govern the fatigue case. Figure 5 below shows that for up to approximately 100,000 cycles, a stress range of 276 MPa is acceptable.

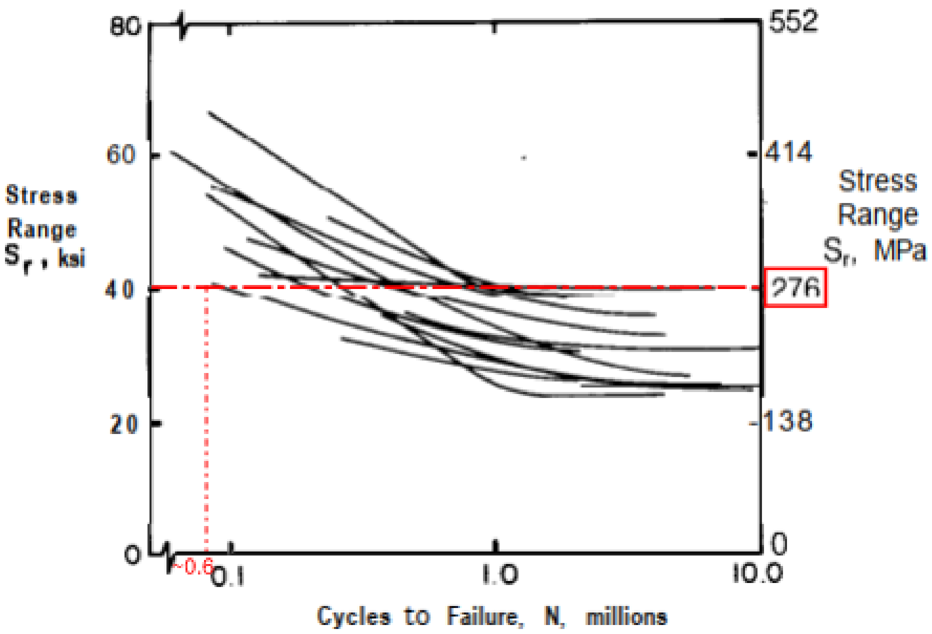


Figure 5: Fatigue stress life curves for reinforcing bars, ACI 215R-6

The concrete shrinkage design has been completed in accordance with NZS 3101:2006, for detailing the external surface reinforcement to minimize crack widths to under 0.3 mm in accordance with CIRIA C766-control of cracking guideline as a limit state for durability and appearance.

8 CONSTRUCTABILITY

Constructability was considered as part of the concrete detailing and specification requirements as summarised in Table 3. The constructability aspects for this project will be further discussed and refined once the Civil works contractor is engaged by the client and their proposed methodology is available.

Table 3: Structural Requirements, recommendations, and modifications to improve constructability.

Factor	Requirement and considerations
Reinforcement size and distribution	The spacing and bar size in this project were designed to optimize both constructability and serviceability requirements. While large-diameter bars at wide spacing can be easier for fabrication, smaller-diameter bars at closer spacing increase the surface area of steel, which is preferable for controlling crack widths.
Reinforcement end returns	Plain round bars were chosen for shorter segments that don't heavily rely on high bond strength along their straight legs for transferring the load, to reduce the hooked end bend diameter. Meanwhile, deformed bars feature a variety of hooked ends at various locations to accommodate fabrication and placement criteria.
Thermal Heat of Hydration	According to the in-house experience of WSP, we have made the following recommendations in our specifications to minimize the generation of heat from the hydration of cement in mass concrete, as outlined below. a) Cement / Binder type and/or additives: GP + Fly Ash b) Minimum cementitious content: 300 kg/m^3 c) Max. water/cement ratio: 0.46 d) Nominal aggregate size: 19 mm e) Slump: 120 mm f) Maximum height of individual concrete lifts is limited to 1.0 m. The design specifications remain performance-based and place the onus on the contractor to demonstrate compliance with the specified requirements.
Interfaces with existing reinforcement	The existing reinforcing is plain round bars throughout, the hooks will be cut off some floor slab reinforcing. Due to the concrete depth and variability in bar size and location, couplers were not considered suitable. A welded solution to new G300 deformed

Factor	Requirement and considerations
	bars is proposed and weldability testing has been completed to confirm suitability.
Formwork considerations	The encasement concrete is curved with a variable perimeter radius and internal void for the turbine pit. To simplify the temporary formwork requirements the internal surfaces of the new concrete have a permanent steel liner, while the external faces are made up of 600 mm wide flat planes to suit typical formwork systems.
Coordination with other services	These types of structures have extensive ancillary equipment, embedded pipework and long-term operational features that need to be incorporated into the detailing. Some items can be adjusted to suit concrete requirements such as pressure sensor ducts and conduits, however others including balancing pipes and major cabling routes must be installed in the designed position. This has required extensive drafting including modelling of primary reinforcing bars in 3D.
Deconstruction considerations	To reduce the construction program and reliability risk, significant deconstruction planning with the demolition contractor (Concut) has been completed. This resulted in a primary deconstruction methodology comprising three work teams working 24-hour operations using concrete wire saws to cut the concrete into 16-tonne blocks which will be removed from the station in specifically designed skips to limit potential damage to the station and surrounding high value equipment.

9 CONCLUSIONS

The redevelopment of Highbank power station poses a unique challenge that has not been previously undertaken in New Zealand. This paper outlines the design considerations and methodologies employed by the design engineers to address the challenges associated with the replacement of the concrete-encased turbine and generator plinth in this project. These challenges encompass soil-structure interaction, the interaction between the existing building with the new concrete elements, the reuse of the existing foundations, and considerations related to vibration, fatigue, and constructability. The reuse of the existing station foundation resulted in savings of over 630 T of CO^2 equivalent in materials alone. WSP are actively involved in collaborating with the contractor for the successful delivery of this project until its completion, with early works underway from January 2024 and civil construction works to commence towards the end of 2024.

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