Design approach for the seismically isolated powder dryer factory at Pahiatua

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ABSTRACT: A 40 m high milk powder dryer tower for the Fonterra plant at Pahiatua is currently being constructed and is planned to start production in August 2015. The dryer is a repeat of the 15 Tonne/hour spray dryer plant commissioned for Fonterra in Darfield in 2012, but located at the Pahiatua site it is subjected to a larger seismic hazard with the nearest fault approximately 6 km away.

This paper describes the base isolated solution; the development of a design philosophy to meet the expectations of the NZ Building Code along with design issues not regularly encountered in non-isolated buildings are described.

1 INTRODUCTION

In mid 2013 Fonterra approved preliminary design for a new 15 tonne per hour spray dryer at the existing Fonterra powder factory site at Pahiatua. The project is being delivered by a design and build team comprising:

- GEA Process Engineering Ltd
- Ebert Construction Ltd
- Silvester Clark Consulting Engineers Ltd
- Compusoft Engineering Ltd

The initial client brief for the proposed Pahiatua plant was to repeat the Darfield D1 dryer plant at the Pahiatua site. It was thought that a "cookie cutter" approach would provide cost and programme certainty to the project based on the existing plant as a "known quantity". A "clean sheet" design was not considered due to the potential programme delay that such a process brought.

The configuration of Darfield D1 comprises a 52.6 m long, 20.4 m wide, 41 m tall building constructed predominantly in reinforced concrete with structural steel framed and clad roof. Seismic resistance is provided by the reinforced concrete suspended floors and steel plate roof acting as diaphragms, and the precast walls and associated columns acting as shear walls. The walls and internal columns are supported on a grid of reinforced concrete foundation beams founding in alluvial gravels. The external views of the building during the construction are presented in Figure 1.

Due to the high (2:1) height to width aspect ratio initial ETABS modelling and response spectra analysis carried out by Compusoft Engineering Ltd indicated that significant uplift forces would occur on the side perimeter foundations. The desire to use shallow foundations for the Darfield No. 1 dryer plant dictated that this was not possible and as such the design actions were derived from a nonlinear response history analysis which allowed for the anticipated rocking of the shallow foundations (Brooke et al. 2012).

The large increase in seismic demand for the new Pahiatua plant (Z=0.42, site subsoil Class "C/D") over that which was originally considered for the Darfield (Z=0.3, site subsoil Class "D") plant meant that a repeat of the Darfield building structure was not practical due to increased uplift of foundations during rocking modes, and increased design actions on diaphragms, shear walls, and foundations. The increases required in floor and wall thicknesses were not practical, and the resulting increase in the stabilising moment due to increased dead load exacerbated the problem.

Figure 1. External view of dryer tower building during construction.

The design solution for the Pahiatua plant was to utilise base isolation which would then allow the existing building design to be repeated. An initial review of seismic isolation indicated that the reduction in base shear would enable the “Darfield” structure design to be used at the Pahiatua site. In addition it was expected that floor accelerations would be reduced to enable the Darfield plant and plant support configurations to be re used without any re-design and strengthening.

2 DESIGN

2.1 Design Philosophy

Compliance with the New Zealand Building Code for the seismic design of a fixed based structure is usually achieved through the use of approved Standards such as NZS1170.5 (2004) and appropriate material Standards. Currently, New Zealand does not have an approved (within the NZBC) Standard for the design of an isolated structure, thus the design falls within the context of a special study in AS/NZS1170.0 (2002). As a consequence the designer is required to decide on what form the special study would take. The approach chosen for this design was to implement the accepted principles of ASCE-7 (2010), Chapter 17. This Standard was chosen as it is relatively mature having had requirements for isolated structures since its 2000 version which are currently being revised for the 2016 release.

The NZBC requires that for the design of a new structure, it shall have; a “low probability of rupturing, becoming unstable, losing equilibrium, or collapsing during construction or alteration and throughout its life”. NZS1170.5 (2004) has been accepted to have satisfied the requirement of “low probability” by requiring that buildings be designed to have some reserve strength over the demands of an ultimate limit state (ULS) event, and by using specified detailing required by the materials standards, it is inferred that there is some resilience against collapse during a larger MCE event. (Fenwick and Dhakal 2007; Almufti and Willford 2013). This is described as a design for “life safety”.

These NZS1170.5 rules are consistent with other Standards around the world. However, when faced with implementing a design for a seismically isolated structure, it becomes immediately apparent that the NZS1170.5 “life safety” philosophy, which designers accept as “advanced thinking” may not be as robust as we have been led to believe. This philosophy has not been developed in a vacuum, but is a pragmatic approach to attempt to achieve a publicly acknowledged satisfactory outcome, taking into consideration the performance capabilities (limitations?) and associated costs of the two dominant structural materials available. For example, the question has to be asked, “if a reinforced concrete building could be built using the same weight of material and same construction cost as today, but with twenty times the strength as the current material, would we be bothering with performing ductile designs for demands from an event that has a 10% chance of occurrence during the lifetime of the structure?” It is most likely the answer would be “NO” and we would design as we do for “gravity”, essentially for strength. The introduction of seismic isolation essentially offers an opportunity to design an “elastic” responding structure for larger earthquakes that maybe designated as maximum credible (MCE).

The pragmatic design approach for this project was to perform a “special study” within NZS1170.5. While specific design to NZS1170.5 is targeted to the ULS, the commentary states that an acceptable probability of collapse is likely to be achieved if the building can be shown to perform “satisfactorily” when subjected to ground motions with a 2% probability of exceedance in the assumed 50 year life. “Satisfactorily” is not defined, but interpreted by the design team to mean that at that demand level, the structure may have little reserve strength but can be shown to be stable. This is consistent with the design approaches contained within modern standards for seismic isolation systems in the U.S. (e.g. ASCE-41 (2014) which define this level of shaking as the Risk-Targeted Maximum Considered Event (MCE_R). The analysis and design of the base isolation system for the Pahiatua dryer building has been undertaken using this level of ground shaking, i.e. ground motions corresponding to the MCE_R derived in accordance with NZS1170.5 (2004) for a return period equal to 2500 years. The resulting design actions have been used without modification in the design of all structural elements.

It is sobering to state, that while the client simply expected that the building meet “code requirements” they may not have had a full appreciation for what this meant, and as it was a design – build contract, the contractors required the design to provide the most cost effective solution, both on material and construction costs. Both of these requirements were met.

2.1.1 *Interpretation of Design Factors*

(i) Structural Performance Factor, S_P

NZ1170.5 scales the demand for the design of ductile buildings through the use of the “Sp” factor. This factor had a controversial introduction into the NZ Loadings Standard (NZS 4203 1992) and has since been modified. It is intended to, empirically, take into account the more robust response of a building structure to an earthquake as compared to that of a single degree of freedom oscillator, in some way accounting for the additional redundancy occurring in a structure. As an isolation system essentially reduces the building’s seismic resistance to one equivalent to that of a single degree of freedom system, a $S_P = 1$ was used to derive the element demands.

(ii) Material Variability

Characteristic material properties form the basis of design of conventional structures. Strength reduction factors further reduce the characteristic strength of structural elements to a dependable strength. In an isolated structure a similar allowance for variability of isolator performance is required, as the isolators have variable material properties and variable construction tolerances as do steel or concrete building elements. However in contrast with standard design, the variability of the isolators properties both through having increased and reduced values from their calculated norms need to be accounted for in design. The reason for this is that higher stiffness and strength transfers through larger forces to the superstructure, while lower values allow for larger isolator displacements and P-delta forces. Taking into account the variability with the isolators is achieved through the use of property modification, or lambda “λ” factors. The property modification factor should be derived with consideration of items that may affect the long term, in-service performance of the system and include items such as manufacturing tolerances, aging, contamination, history of loading, temperature and other effects. The differing hysteretic loops for both the upper bound and lower bound scenario are illustrated in Figure 2. Typically lower values of displacement, D and damping, β are associated with the upper bound scenario whereby λ_{max} is used to scale Q_d and K_d. The use of lambda factors is commonplace in the design of bridges and is also incorporated into ASCE-41 (2013). It is expected that the next revision to ASCE-7 (2010) will follow a similar approach.

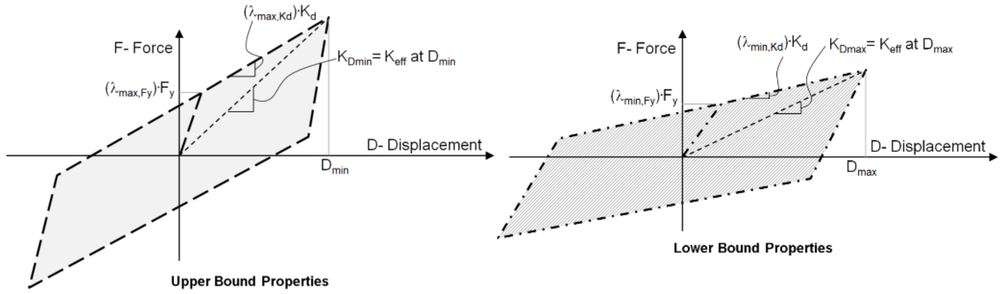

Figure 2. Hysteretic Loops for Upper and Lower Bound Properties.

As the demands on the superstructure were determined for a level of shaking corresponding to the MCE_R, the design of the elements were undertaken using a strength reduction factor, ϕ equal to “1”. We comment, that while for the ease of peer review considerations, the design of the concrete members used characteristic material properties, we would contend that mean or expected material values would be more consistent with performance based design philosophies.

2.2 Isolator scheme design

During the initial stages of the project it was established that the primary design goal was to achieve a solution which made the best use of the available strength in the superstructure on the basis that this was not intended to be changed from that previously used in Darfield. From this, the key driver in specifying the bearing properties was to minimise the displacement demands at the isolation level, thus maximising the design efficiency of the superstructure and minimising any perceived financial implications on the construction budget as a result of the cost of “large” isolators, along with the associated costs due to the size of the plinths, and earthworks costs as a result of the increased basement extents.

2.2.1 *Isolator properties*

Early in the design process, Earthquake Protection Systems (EPS) were selected as the preferred supplier for a friction pendulum type isolation bearing system which was suitable for this project.

In order to achieve an acceptable balance between base shear demand and isolator displacement a coefficient of friction equal to 0.1 was selected for the isolators. While the selected friction coefficient could be considered quite high and potentially lead to “high” in-structure accelerations (though still markedly lower than an equivalent design that did not utilise isolation) this was deemed acceptable in the context of the project on the basis that the plant design was unlikely to be adversely affected. The adopted bearing design has a displacement limit equal to 912 mm before engaging the perimeter stops. A schematic of the adopted bearing is presented in Figure 3 below.

As previously discussed, it is widely recognised that there is a need in the design of seismically isolated structures to consider a lower and upper bound value for the basic mechanical properties though the use of property modification factor (λ). For friction pendulum type bearings as adopted for use in this project, the “yield” force, Q_d is directly related to the friction coefficient of the sliding surface whereas the post “yield” slope, k_d is directly related to the radius of the sliding surface. The radius of the sliding surface is manufactured to close tolerances and as such is it is not expected that there will be much (if any) variability in the post yield slope and therefore only variability in the coefficient of friction is required to be considered (Constantinou et al. 2007). The lower and upper bound property modification factors, λ_{min} & λ_{max}, adopted for use in the project were recommended by the bearing supplier (EPS) and are presented in Table 1 below.

Table 1. Property modification factors (λ).

Parameter	λ_{min}	λ_{max}
Friction coefficient, μ	0.8	1.3

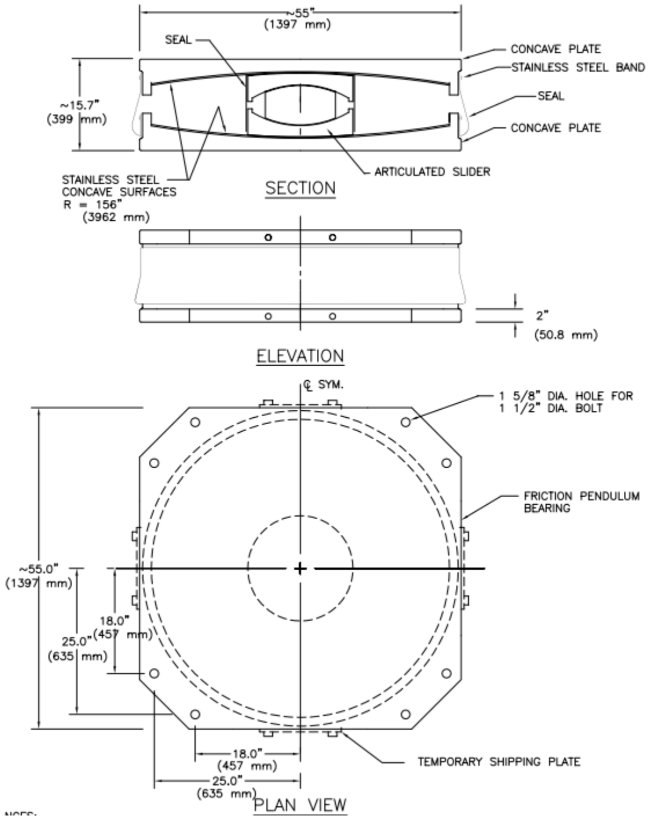

Figure 3. EPS Triple Pendulum Bearing (TPB) Schematic.

3 SEISMIC ANALYSIS

3.1 Seismic hazard

A geotechnical investigation was carried out in order to determine the site subsoil class with respect to NZS1170.5 (2004). Analysis of the shear wave velocity data indicated that the natural period for the site lies somewhere between that applicable for site subsoil Class C and Class D and as such a modified spectral shape factor was derived following the recommendations of McVerry (2011). The 5% damped elastic site spectral demand for the MCE_R derived in accordance with the requirements of NZS1170.5 (2004) is presented in Figure 4 below.

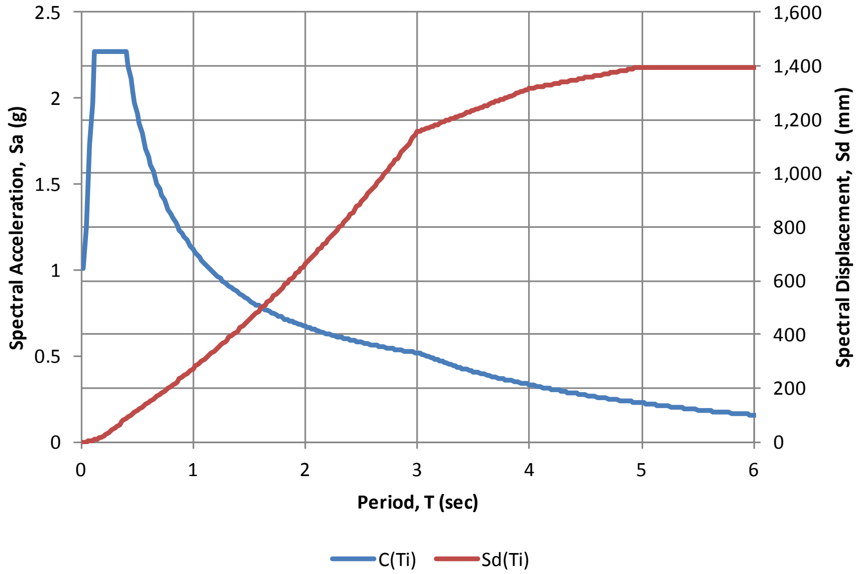

Figure 4. MCE_R demand spectra (5% damped).

3.2 Equivalent Lateral Force procedure

In order to benchmark the results of a nonlinear response history analysis the isolator demands were first evaluated via the Equivalent Lateral Force (ELF) procedure from ASCE-7 (2010) with consideration of the lower and upper bound isolator properties. The calculations for the ELF procedure are presented in Table 2 below.

Table 2: Isolator demands determined via the ELF procedure

Parameter	Nominal	Lower	Upper	Units
$\lambda(Q_d)$	-	0.8	1.3	-
Characteristic strength factor, Q_{df}	0.1	0.08	0.13	-
$\lambda(K_d)$	-	1.0	1.0	-
Effective Radius, R_{eff}	7.645	7.645	7.645	m
Post Yield Stiffness Factor, K_{df}	0.10	0.10	0.09	-
Isolator Displacement, D	0.741	0.796	0.675	m
Maximum Base Shear Coefficient, C	0.20	0.18	0.22	-
Effective Period, T_{eff}	3.89	4.17	3.53	sec
Site Spectra Acceleration, $S_a(T_{eff}, \xi=5\%)$	0.35	0.31	0.40	g
Viscous damping ratio, ξ_{eff}	0.32	0.28	0.38	-
Damping Factor, B_L	1.75	1.67	1.84	-

It can be seen from Table 2 that with the adopted bearing properties, it is likely that significant levels of hysteretic damping can be achieved which in turn reduces the displacement demand from that implied by Figure 4. From this it was able to be determined with reasonable confidence that the selected bearing would likely possess enough displacement capacity for the MCE_R event, and also the Darfield plant superstructure would likely have sufficient strength to resist the imposed base shear. With this knowledge, the final design verification could be undertaken via the non linear response history procedure.

3.3 Non-linear Response History Analysis

3.3.1 *Overview*

The final verification of the design of the superstructure and the isolation elements was carried out based on the results of a non-linear response history analysis. The analysis was undertaken with consideration of NZS1170.5 (2004) and ASCE-7 (2010), along with the relevant NZ material Standards (e.g. NZS3101 (2006)). Key items of interest are discussed in the following sections.

3.3.2 *Ground motions*

A suite of seven, three component earthquake ground motion records were provided by GNS Science (2013) for the site which were suitable for use in a nonlinear response history analysis for the MCE_R scenario and are presented in Table 3 below.

Scaling of the ground motion records was undertaken in accordance with the general requirements of ASCE-7 (2010). The period range of interest was taken as $0.5T_{D,min}$ through to $1.25T_{D,max}$ where $T_{D,min}$ & $T_{D,max}$ are defined as the effective periods of the isolated structure determined via the ELF procedure for the upper and lower bound properties respectively, and considering the MCE_R demand spectrum as defined previously.

Table 3. Selected ground motion records

Accelerogram	M_W	Distance	Mechanism	Type of event	Site Description
Duzce (Turkey) 1999	7.2	8.0	Strike-slip	-	Soft Soil
El Centro Imperial Valley (California) 1940	7.0	6.0	Strike-slip	-	Deep Soil
Tabas (Iran) 1978	7.4	1.2	Strike-slip	Strong Forward Directivity	Sedimentary rock
TCU051 Chi-Chi (Taiwan) 1999	7.6	7.0	Reverse	-	V_s30=273m/s
TCU050 Chi-Chi (Taiwan) 1999	7.6	9.0	Reverse	-	V_s30=273m/s
HHKD085 Hokkaido (Japan) 2003	8.3	46.0	Subduction Interface	-	Medium Soil
PS10 Denali (Alaska) 2002	7.9	2.7	Strike-slip	Strong Forward Directivity	V_s30=330m/s

3.3.3 *Analysis model*

A three-dimensional finite element analysis model of the building above plinth level was created in the commercial software package ETABS 2013 (CSI Berkeley 2013). In this model the superstructure was modelled using linear elastic frame and shell elements in accordance with the structural drawings. Stiffness modification factors were applied to all concrete elements in order to allow for the level of cracking that is anticipated when they attain their ultimate strength. All nonlinear behaviour was confined to the isolators which were modelled using non-linear link elements which support the ground floor beams. Figure 5 below shows an extruded 3D view of the ETABS object model.

A series of nonlinear response history analyses were undertaken via the Wilson Fast Non-linear Analysis (FNA) method using the ground motion records identified in Table 3. In these analyses the damping was taken as constant and equal to 5% for all “elastic” superstructure modes and zero for any modes associated with the isolators. In accordance with the provisions of ASCE-7 (2010) the design member forces and deformations throughout the structure have been taken as the average of the peak responses determined from each of the response history analyses.

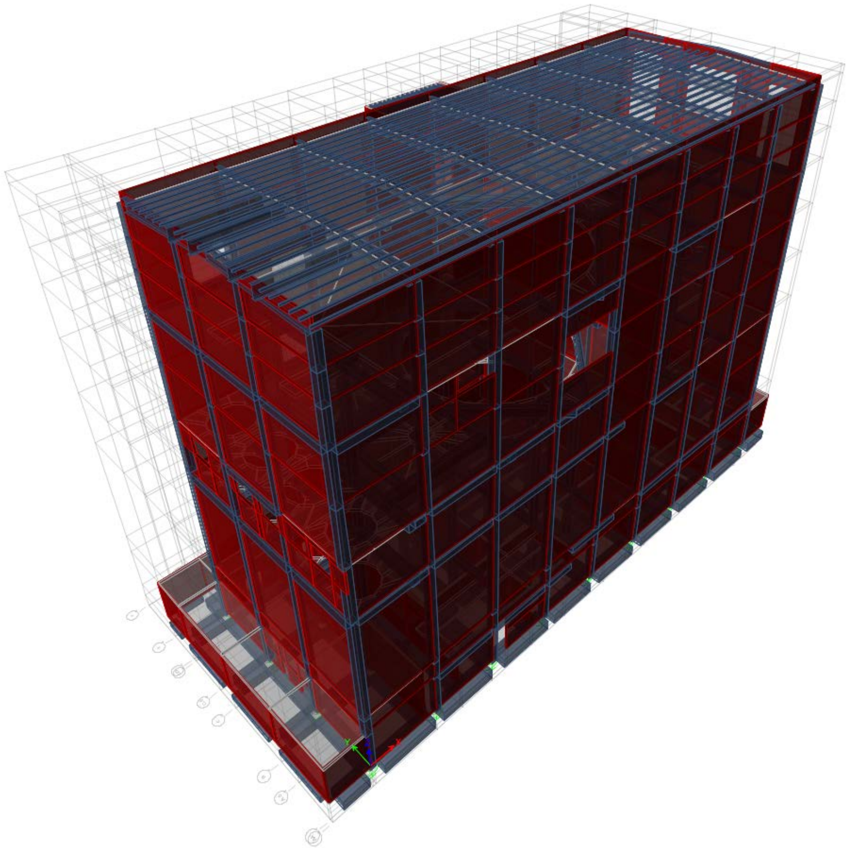

Figure 5. 3D view of ETABS object model.

4 DESIGN AND CONSTRUCTION IMPLICATIONS RELATING TO IMPLEMENTATION OF SEISMIC ISOLATION

4.1 Foundations

The foundation system comprises a 1200 mm thick reinforced concrete raft foundation supporting 1600 mm square plinth columns 1200 mm high. The plinths in turn support the EPS triple pendulum bearings. The ground floor comprises a two way grid of reinforced concrete beams, typically 1200 mm deep x 1500 mm wide supported directly on the bearings. The beams support floor slabs and the main superstructure precast walls. The principal superstructure columns are supported directly over the bearings. Refer to Figure 6 below for arrangement of raft foundation, plinth, bearing and ground floor.

The design of the plinths was initially driven by the practical consideration of bearing size (1400 mm square) and the allowance for access to the sub floor space for maintenance. Design of the plinths also needed to consider the following criteria:

- Axial loading (both static and earthquake induced),
- Direct seismic shear and resulting flexure,
- P-Δ effects due to the axial load and bearing displacement.

The P-Δ effects were of a significant magnitude with the total bending moment in the plinths equalling approximately two times the flexure calculated from direct shear x effective plinth height alone.

The design of the ground floor beams required combination of the following effects:

- Gravity and live loading from ground floor and walls,
- Ground floor diaphragm actions,
- P-Δ effects from axial load reactions and bearing displacements.

It should be stressed that for a triple pendulum bearing with both upper and lower bearing surfaces concave, the P-Δ effects are shared between the supporting plinths below and the super structure above. Hence the effects need to be considered in the both cases.

The raft foundation was chosen to achieve the following outcomes:

- The raft covering the full building area achieved imposed bearing pressures and settlements in line with alluvial gravel subgrade capacities,
- The raft mitigates the risk of sub surface liquefaction under seismic loading at or above DBE (500 year return period) events.

The thickness of the raft was governed by the plinth column axial and flexure induced punching shear. The 1200 mm thickness means that vertical shear reinforcing is not required.

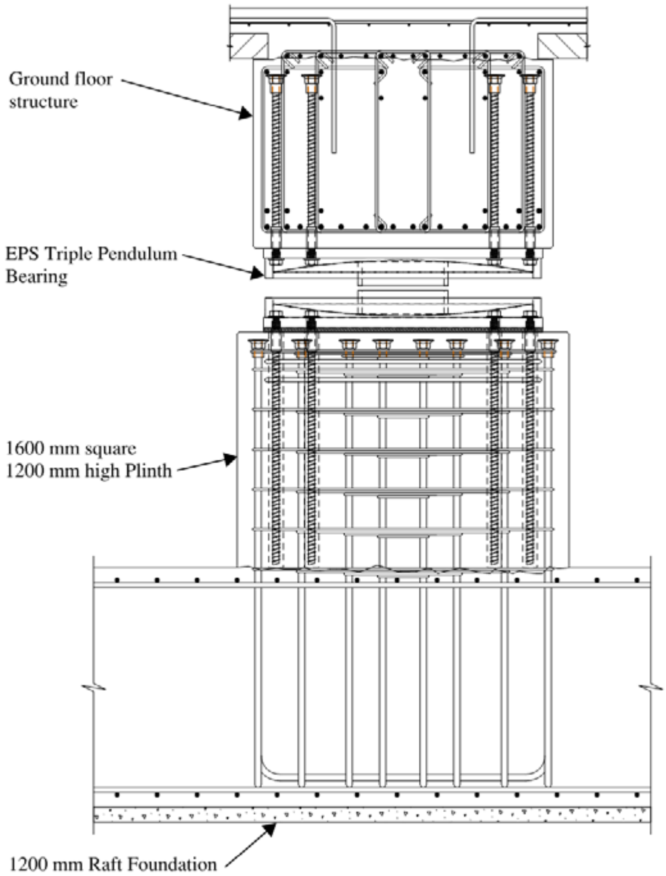

Figure 6. Foundation raft, plinth, EPS triple pendulum bearing, and ground floor detail.

4.2 Bearing bolted fixings to the plinths

If cast in holding down bolts are to be used consideration should be given to tolerance requirements for the plan set out of the bolts and levelling of the bearings. For the Pahiatua project corrugated ducts were cast into the plinths, and the holding down bolts/anchor bars were prefixed to the bearings. The assembly was then lowered into the grout filled duct. The same high strength shrinkage compensated grout was used as a bedding mortar between the plinth and underside of the cast steel bearing base. Site photos taken during the bearing installation are presented in Figure 7.

Figure 7. EPS bearing installation

4.3 **The ground floor structure**

The two way grid of beams forming the ground floor frame was cast insitu on a false work platform covering the full building plan area. Because the building grid layout was not regular, the placing of the beam reinforcing steel was very slow and delayed the programme significantly. In future it is considered that a flat slab replicating the founding raft would be a better solution for the ground floor. While more concrete would probably be necessary, the time saving in placing raft reinforcing and less formwork would more than offset the added materials costs.

4.4 **The basement**

The depths of ground floor structure, bearings, plinths, and raft mean that the raft is founded approximately 4.0 m below ground level, well into the alluvial gravel layer with the unsuitable clays overlying the gravels excavated. Refer to Figure 8 for a cross sectional view.

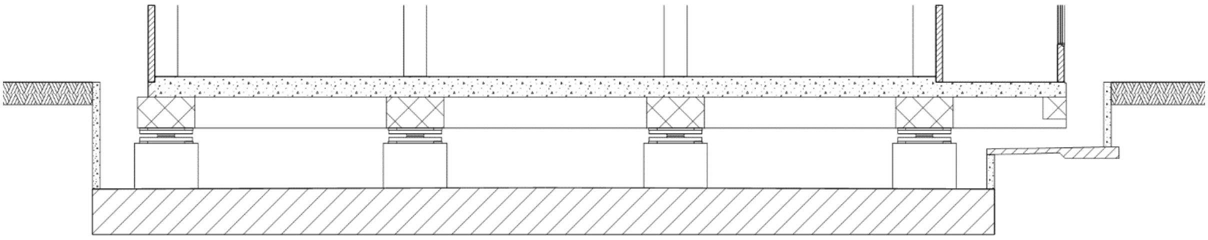

Figure 8. Cross section through raft foundation, plinths, bearings and ground floor.

The moat walls are constructed with 980 mm clearance from the dryer superstructure. This allows the relative displacements anticipated when the building is subjected to the MCE_R to be achieved before the building contacts the moat walls.

4.5 **Plant/services interface**

Piped services cross the seismic separation via an articulated pipe bridge “loop”. This is similar to the thermal expansion loops seen on oil or thermal steam pipe lines.

It is intended that the services are fully operational under displacements expected for a Serviceability Level Earthquake (SLE), and operational or readily repairable for displacements expected for a Design Basis Earthquake (DBE), depending on risk to building occupants from failure of a particular service line.

5 BUILDING COSTS

Table 4 below presents the relative costs of each component in both the Darfield and Pahiatua plants. Total project and building costs are commercially sensitive and confidential to Fonterra and the project contractors, and as such only the percentage costs normalised against the total cost of the original Darfield plant are presented.

It is apparent from Table 4 that through the use of seismic isolation, the total project cost only increased by some 2% overall vs the Darfield design. This is despite the fact that the Pahiatua site has significantly higher seismicity than that of Darfield, and the new design is expected to result in zero damage to the structure and its contents for seismic events that have a very low probability of exceedance. The construction of an equivalently specified plant at this location without the use of base isolation would necessitate a complete redesign of the building structure and plant elements to adequately accommodate the approximate 40% increase in demands between the two sites. The commensurate increase in project cost for this design approach has not been assessed however it is expected to be significant.

Table 4. Relative project costs

Cost Component	Darfield	Pahiatua
Plant	80%	80%
Structure	20%	22%
Foundations	0.4%	2.2%
Total Project	100%	102%

It should be noted that if the “cookie cutter” approach had not been requested by the client, it is likely that with the implementation of seismic isolation, further savings could have been made in the design of the new Pahiatua plant and structure. These savings could have been achieved through:

- The reduction of lateral strength of the superstructure;
- The reduction of the lateral strength of the plant equipment and associated fixings due to the potential for significantly reduced floor spectra;
- Furthermore, as a result of the significantly improved performance and reliability present through the use of seismic isolation, an assessment of the lifecycle costs would likely show significant cost savings over the longer term.

6 CONCLUSION

A seismically isolated building for the new Fonterra spray dryer project at Pahiatua has been designed and is currently in construction. The design philosophy and design/construction issues have been explained. It has been found that there were significant cost savings using this approach which allowed the re-use of an existing design solution originally developed for the Darfield dryer plant building structure and process plant.

7 CONCLUDING COMMENT

At the early stages of the design process the client and contractors were not concerned with post earthquake serviceability. Only after the design process had been completed have the benefits of the base isolation system been demonstrated to the client.

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