DESIGNING FOR SEISMIC EVENTS – A CASE STUDY

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ABSTRACT

This paper presents a case study of the seismic design for a multi-level car park constructed over soft Holocene alluvial deposits in Brisbane. The paper presents the design approach and the structural and geotechnical analysis undertaken to assess the earthquake induced impacts on the structure to demonstrate the adequacy of the proposed foundation design.

The paper highlights the following design aspects:

1) The requirements of Clause 5.2.2 of the Australian Standards AS1170.4:2007 to provide tie beams may not be appropriate in all cases.
2) The use of elastic response spectrum analysis using modal superposition methods can result in refined inertia forces on the foundation compared to those computed using pseudo-static methods.
3) The earthquake-induced kinematic effect should not be overlooked with respect to total deformation of the foundation.

1 INTRODUCTION

Australian Standard AS 1170.4 – 2007 provides the designers of structures with earthquake actions and general detailing requirements for use in the design of structures subject to earthquakes. Whilst AS 1170.4 – 2007 provides detailed guidance on how the designer must consider earthquake effects on the superstructure, guidance on how the designer may consider the ground response and its impact on the structure during an earthquake event is less clear.

This paper presents a recent case study of the foundation design for a multi-level car park constructed over soft Holocene deposits in Brisbane. The structural designer, working closely with the geotechnical engineer, was able to develop an alternative, cost-effective foundation solution that complied with the requirements of the Building Code of Australia (BCA) (2007 version) and removed the need for tie beams, as stipulated by AS1170.4 – 2007 for the ground conditions present at the site.

2 CODE DESIGN REQUIREMENTS

2.1 BUILDING CODE OF AUSTRALIA

The Building Code of Australia (BCA) is produced and maintained by the Australian Building Codes Board on behalf of the Australian Government and each State and Territory Government. The BCA is a uniform set of technical provisions for the design and construction of buildings and other structures throughout Australia whilst allowing for variations in climate and geological or geographic conditions. A building solution will comply with the BCA if it meets the Performance Requirements, which can only be achieved by:-

(a) Complying with the Deemed-to-Satisfy Provisions or
(b) Formulating an Alternative Solution which –
    (i) Complies with the Performance Requirements or
    (ii) Is shown to be at least equivalent to the Deemed-to-Satisfy Provisions or
(c) A combination of (a) and (b).

BCA allows alternative solutions to be sought for the Deemed-to-Satisfy provisions, using the following Assessment Methods, or any combination of them:

(a) Evidence to support that the use of a material, form of construction or design meets a Performance Requirement or a Deemed-to-Satisfy Provision.
(b) Verification Methods such as –
    (i) The Verification Methods in the BCA or
    (ii) Such other Verification Methods as appropriate authority accepts for determining compliance with the Performance Requirements.

(c) Comparison with the Deemed-to-Satisfy Provisions.
(d) Expert judgement.

The relevant Performance Requirements of BCA Clause BP1.1 are as follows:

A building or structure, to the degree necessary, must—
(i) remain stable and not collapse and
(ii) prevent progressive collapse and
(iii) minimise local damage and loss of amenity through excessive deformation, vibration or degradation and
(iv) avoid causing damage to other properties by resisting the actions to which it may reasonably be subjected.

The actions to be considered to satisfy the above include earthquake action.

2.2 AUSTRALIAN STANDARDS AS1170.4 – EARTHQUAKE LOADING CODE

The most recent version of the earthquake code, AS1170.4 - 2007 Structural design actions Part 4: Earthquake actions in Australia, was adopted in the structural design of the multi-level car park. The latest version of AS1170.4 represents a refinement and progression in the approach for determining earthquake actions for building structures in Australia.

The updated code includes a subtle but fundamental change to the detailing requirements for all buildings. Under the updated code general design and detailing principles are adopted for all earthquake design categories and these are outlined in *Clause 5.2 Basic Design Principles*. The design procedure to be undertaken is set out in *Clause 2.2 Design Procedure* and presented in *Figure 2.2* of the updated code, which is reproduced as Figure 1 for ease of reference. It can be seen from this diagram that all design categories (EDCI to EDCIII) require the application of *Clause 5.2*.

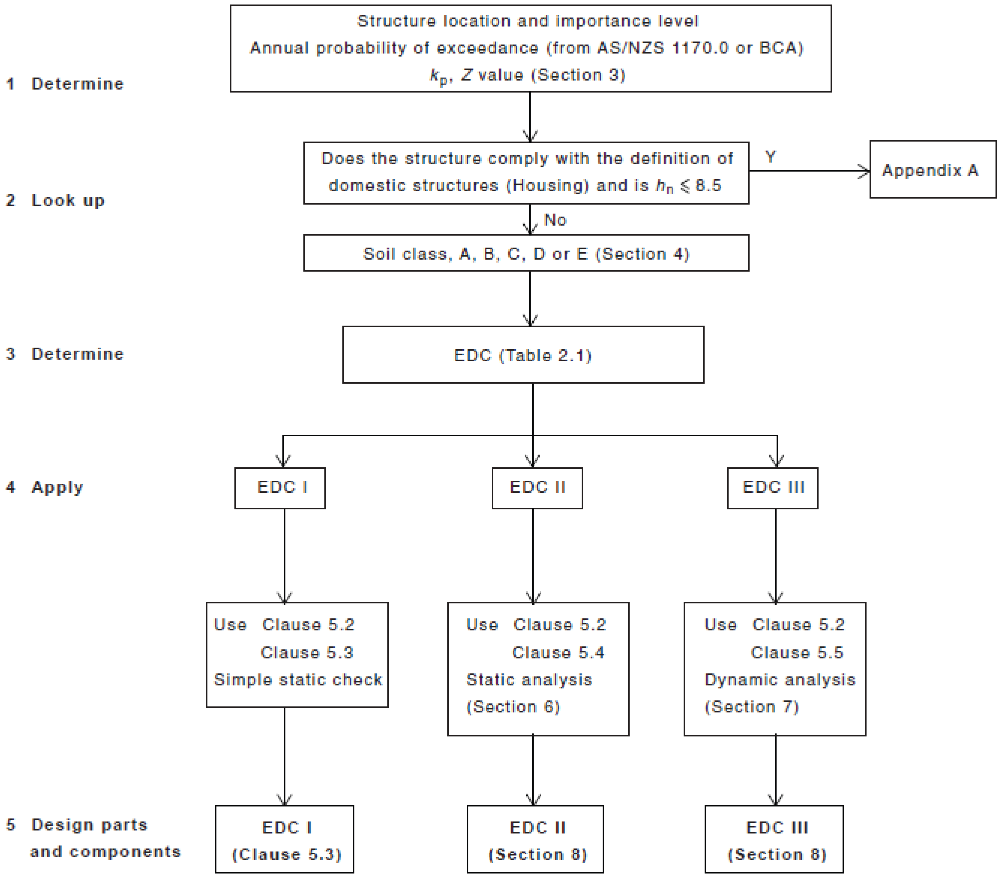

Figure 1: Flow Diagram – Design Procedure

In *Clause 5.2*, *sub-clause 5.2.2* addresses the requirement for tying the structure together and is as follows:

5.2.2 Tying structure together

All parts of the structure shall be tied together both in the horizontal and the vertical planes so that forces generated by an earthquake from all parts of the structure, including structural and other parts and components, are carried to the foundation.

Footings supported on piles, or caissons, or spread footings that are located in or on soils with a maximum vertical ultimate bearing value of less than 250kPa shall be restrained in any horizontal direction by ties or other means, to limit differential horizontal movement during an earthquake.

The intent of AS1170.4 performance criteria is to prevent collapse of the structure during an earthquake event. Therefore the intent of *Clause 5.2.2* is to ensure adjacent columns are limited from displacing differentially during an earthquake and leading to a collapse of the structure.

2.3 DISCUSSION OF CODE REQUIREMENTS

Tying footings is a common requirement in many earthquake design codes (e.g. NZS1170.5, UBC 1997, IBC 2009 and EC8), though there is little guidance given on how this requirement relates to the site ground conditions. There is a general consensus across the various codes that where the site sub-soil classification is deemed to be rock, then tying is not required.

AS1170.4- 2007 stipulates an empirical design rule regarding the requirement to tie footings together which is related to the ultimate vertical bearing capacity of the material on/in which the footings are founded. The ability of the ground to provide lateral restraint to a pile cap or spread footing is not directly related to the ultimate bearing capacity of the foundation soils, rather this value is a guide to the nature and quality of the ground. This is particularly the case for a site where the vertical bearing capacity of the foundation soils is largely governed by the presence of soft clay underlying sand at depth.

In order to meet the Deemed-to-Satisfy requirements of BCA, the requirements of AS1170.4-2007 Clause 5.2.2 must be met. For the ground conditions prevalent at the multi-level car park site, the ultimate bearing capacity of the foundation soils was less than 250kPa and this resulted in the requirement to tie the pile caps in accordance with AS1170.4.

It was considered that under BCA an alternative solution could be sought for the multi-level car park foundation that complied with the Performance Requirements and did not require the use of tie beams.

3 STRUCTURAL DESIGN APPROACH

3.1 INTRODUCTION

The new multilevel car park is a nine storey building approximately 250 m long and 75 m wide. The height from ground to the last concrete floor at Level 9 is 24.2 m with overall height to the steel roof of approximately 27.5m. The structure adopts a moment resisting frame as the structural system for resisting lateral loads. The lift and stair cores do not carry any component of lateral load and are structurally isolated from the floor plates by isolation joints. The structural system comprises reinforced concrete columns supporting a one-way post tensioned beam and slab arrangement. The structural grid is approximately 10.4 m by 8.3 m to 8.9 m to suit parking bays.

Column sizes are typically 1000 mm x 400 mm for Levels 1 to 4 and 900 mm x 400 mm for Levels 5 to 9. Concrete strengths vary from N60 (Level 1 and 2) to N40 (Level 3 to 9). The band beams are generally 1500 mm / 1800 mm wide by 325 mm deep and the slabs are typically 150 mm thick for internal panels and 190 mm for the cantilever edges. The beams are 400 mm deep at the expansion joints.

Due to the overall length of the floor plate, expansion joints are provided at approximately 90 m centres with intermediate construction joints at 30-40 m. A nominal joint gap allowing for thermal expansion or movement due to wind load up to 20 mm is provided. The joint is not intended to transfer lateral loads between adjacent sections of the building. The dowels across the joint are primarily designed to control differential movements each side of the joint due to in-service gravity loads. The dowels are ignored in the lateral analysis and the building is broken into three separate sections that are analysed and designed to be self supporting.

The foundation system adopted for the project comprises driven precast piles founded approximately 25-30 m below ground level. The structural analysis was performed using the structural analysis program ETABS v9.6 by Computers and Structures Inc. The structural analysis modelled the foundations as equivalent springs.

3.2 STRUCTURAL ANALYSIS

During the concept design a conventional pseudo-static approach was adopted to allow conceptual sizing and assessment of the cost data of the structural elements to be completed. It was clear from the outset that earthquake loading was critical to the design of the structure. A comparison of the base shears and overturning moments showed that earthquake was the dominant case, being some 65% greater than the robustness load and 177% greater than the wind load. It was also evident that the pseudo-static approach to earthquake load derivation predicted a 30% larger earthquake load than if an elastic response spectrum analysis using superposition techniques was undertaken.

To refine the earthquake design load an independent seismic hazard assessment was undertaken and a site specific response spectrum for ground motion was developed. The purpose was to use this in conjunction with an elastic response spectrum analysis of the structure in accordance with Section 7 of AS1170.4 to determine appropriate design actions. However the design was completed using the code-specified response spectra for ground motion, since the site specific response spectrum was received part way through the design process. The site specific response spectrum provided a slightly improved spectrum for the site when compared to the code spectrum, though structurally there was little benefit in re-analysing for minimal gain. In essence the site specific spectrum verified the assumptions for ground motion adopted in design.

3.2.1 Design Process

Accurate simulation of the built form is critical for an accurate response spectrum analysis to be performed since assumptions made in analysis directly affect the results. Seemingly conservative assumptions that are appropriate for pseudo-static analysis, as used for wind load design or static earthquake design, can lead to an underestimation of possible design actions determined by a modal analysis. A pseudo-static analysis only considers the first two translational modes and the code procedure limits the calculated base shear to a minimum of 80% of the base shear determined from a simplified method of determining structure period. The earthquake code allows the use of an elastic response spectrum analysis using modal superposition methods to determine the structure response taking into account the mass and stiffness distribution and in turn to predict the design actions due to earthquake. The multilevel car park is quite a flexible structure and its fundamental period falls at the tail end of the code spectrum for ground motion and results in an earthquake load prediction well below that predicted by a pseudo-static assessment.

To try and ensure an accurate mathematical model of the structure was adopted for design the following process was followed.

- Step 1 – Sensitivity analysis - determine the sensitivity of structure performance to various structural and foundation parameters.
- Step 2 – Assess benefits of adopting an elastic response spectrum analysis
- Step 3 – Refinement of the three dimensional model – a first run analysis to assess appropriate levels of cracking in the columns beams and slabs.
- Step 4 – Final analysis – determine the design actions for the foundations, columns, beams and slabs.

3.2.2 Step 1 - Sensitivity Analysis

A detailed parametric study of representative frames of the structure was performed to determine the sensitivity of varying a number of structural parameters on the prediction of earthquake design actions. A two dimensional linear elastic analysis was adopted for the sensitivity analysis. The parameters studied included column stiffness, and beam and slab stiffness.

A similar, separate study of the effects of foundation stiffness was also performed to assess the effect of the various components of foundation stiffness on structure response. The components of foundation stiffness studied were horizontal and vertical stiffness and rotational stiffness. The process involved collaboration with the geotechnical engineer and the piling subcontractor to determine appropriate design input parameters.

To assess the sensitivity of the structure performance to the various parameters the following performance measures were compared, including structure period, displacement of the top floor, column bending moment, column axial load, beam bending moment, beam shear force, slab bending moment and slab shear force. Since earthquake is the most critical design load case, the structure period is the most important performance measure as the design load is derived from this parameter. The displacement of the structure and the column base bending moment provide simple measures of the physical impacts on the structure and in particular the foundations. To demonstrate the influence of the various parameters on structure response the results are compared in Figures 2, 3 and 4, for the primary beam frames for an arbitrary set of lateral loads based on a pseudo-static earthquake load. The recommended model results are also plotted. The choice of appropriate structural properties and foundation stiffness for the recommended model was determined to ensure the maximum column base actions would be derived for pile and column design and also provide maximum elastic displacement at the top of the building.

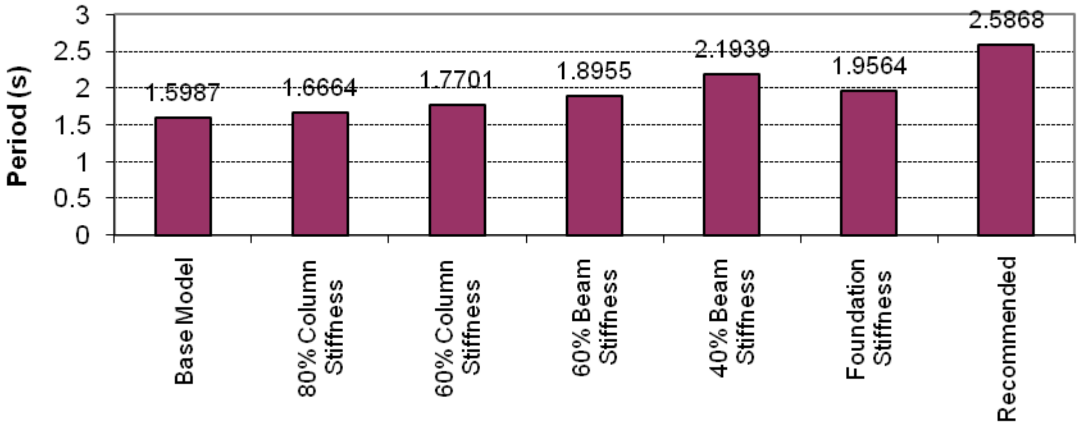

Figure 2: Effect on Structure Period (X Translation)

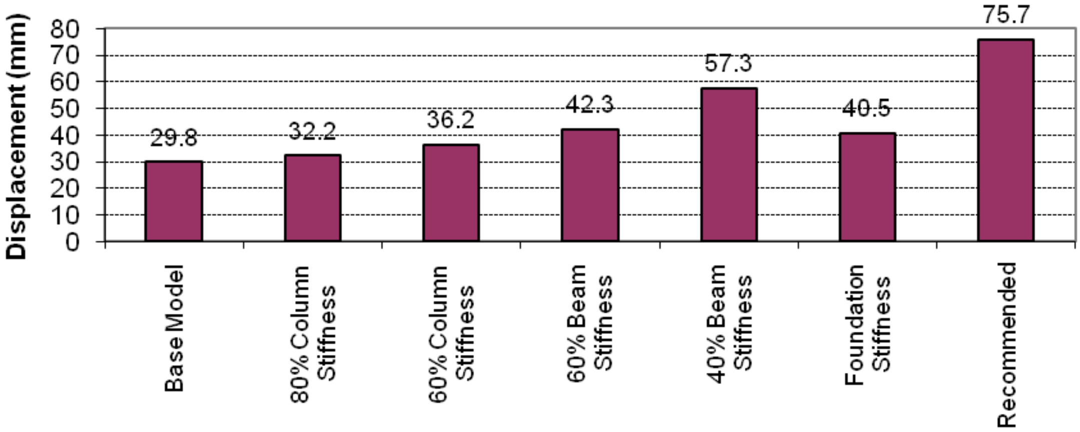

Figure 3: Effect on Displacement at the Top Floor (X Translation)

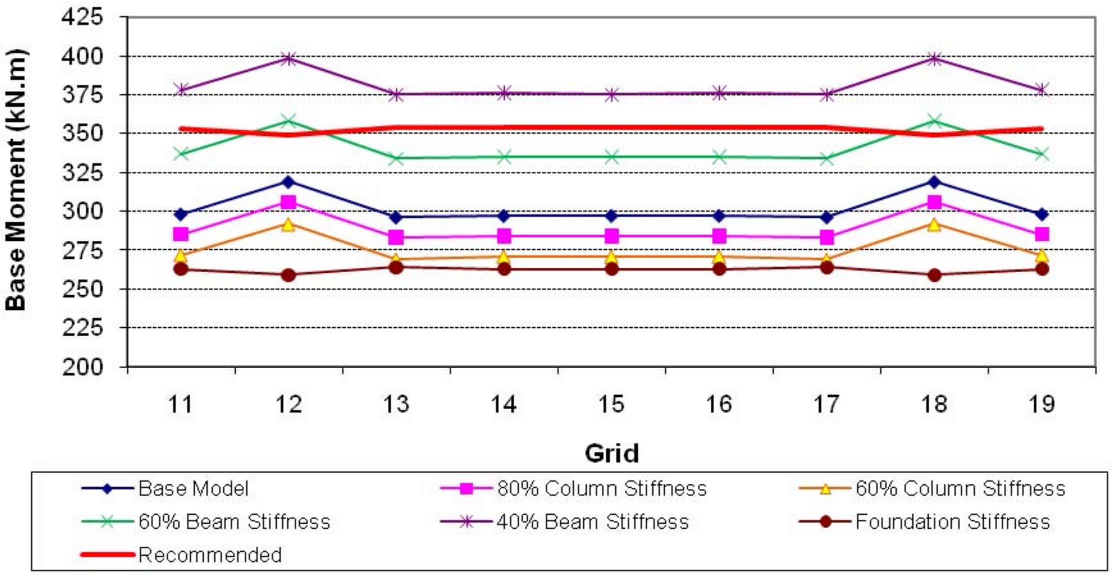

Figure 4: Effect on Column Moment (X Translation)

A separate study of the influence of the various components of foundation stiffness was also performed. Representative column actions derived from a pseudo-static analysis were provided to the piling designer. The piling designer analysed the representative foundations using PIGLET and provided associated foundation deformations. The deformations were used to determine equivalent spring stiffness values for adoption in the structural model. To demonstrate the influence of the various parameters on structure response the results are compared for the primary beam frames for the same arbitrary set of lateral loads in Figures 5, 6 and 7.

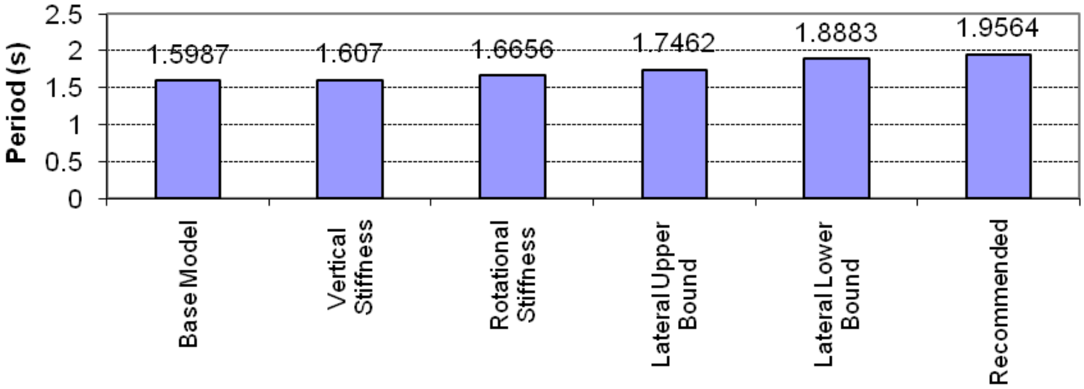

Figure 5: Effect of Foundation Stiffness on Structure Period (X Translation)

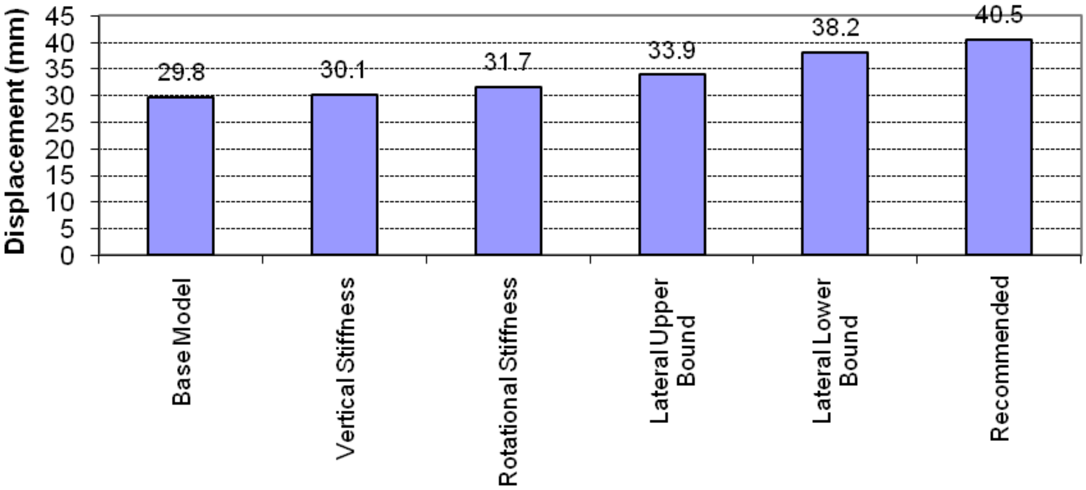

Figure 6: Effect of Foundation Stiffness on Displacement at the Top Floor (X Translation)

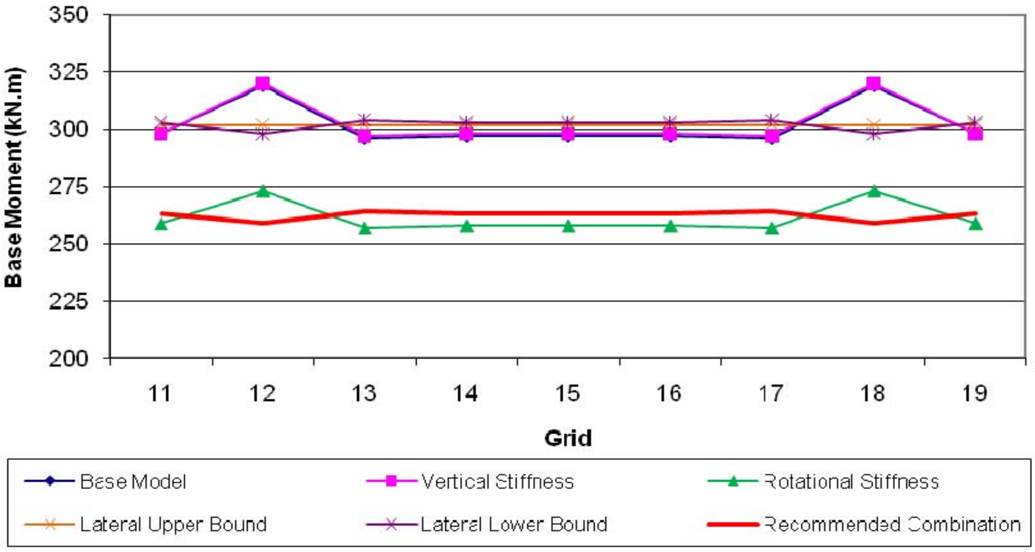

Figure 7: Effect of Foundation Stiffness on Column Moment (X Translation)

The sensitivity analysis showed that foundation stiffness had a significant effect on structure response and in particular rotational stiffness.

Since the primary objective was provide a performance based solution to the requirement for tie beams between the foundations the design actions most significant to this assessment are at the base of the columns i.e. base shear and bending moment. As can be seen from the above figures all reductions in structure stiffness due to cracking of columns, beams or slabs result in a longer period for the structure. A longer period would reduce the column actions. Therefore in order to estimate the maximum column base shears in the first run three dimensional analysis gross properties for all structural elements was adopted.

3.2.3 Step 2 – Elastic Response Spectrum Analysis

Figures 8 and 9 show the influence of the various structural parameters investigated on the base shear for the two orthogonal directions of the structure determined using an elastic response spectrum analysis. The predicted base shear for wind load and a pseudo-static earthquake load are also plotted for comparison.

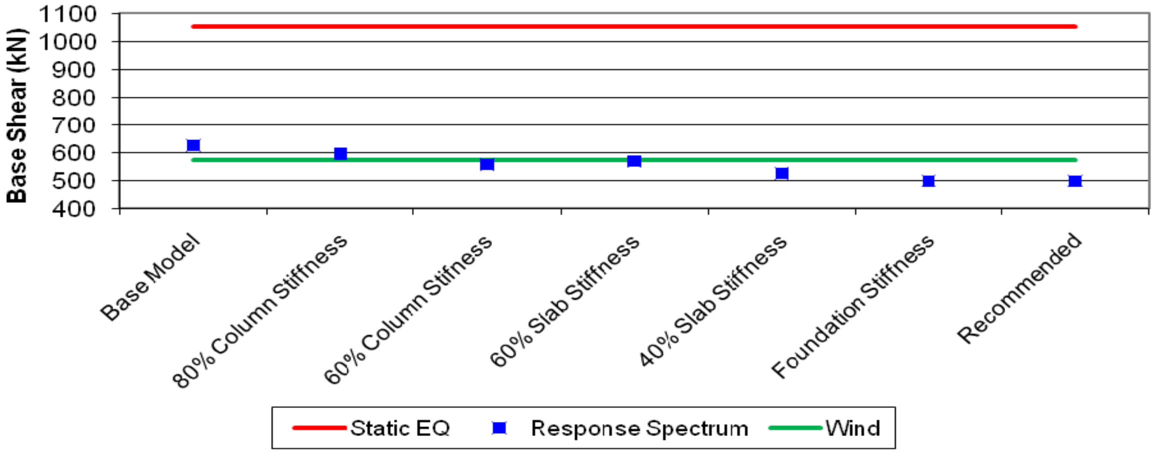

Figure 8: Effect on Design Base Shear – Y Translation

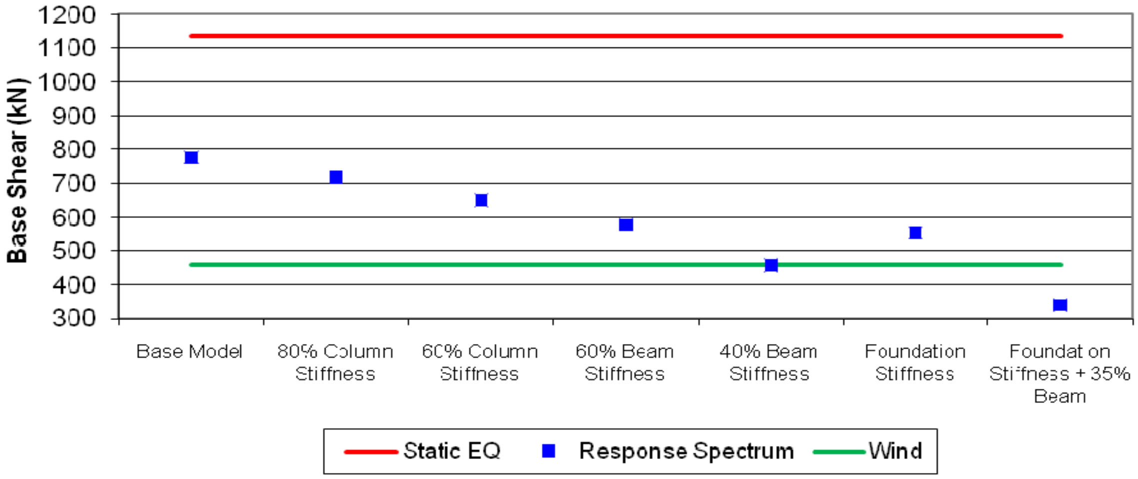

Figure 9: Effect on Design Base Shear – X Translation

Figures 10 and 11 show the influence of the various foundation stiffness parameters investigated on the predicted base shear for the two orthogonal directions of the structure. The predicted base shear for wind load and a pseudo-static earthquake load are also plotted for comparison.

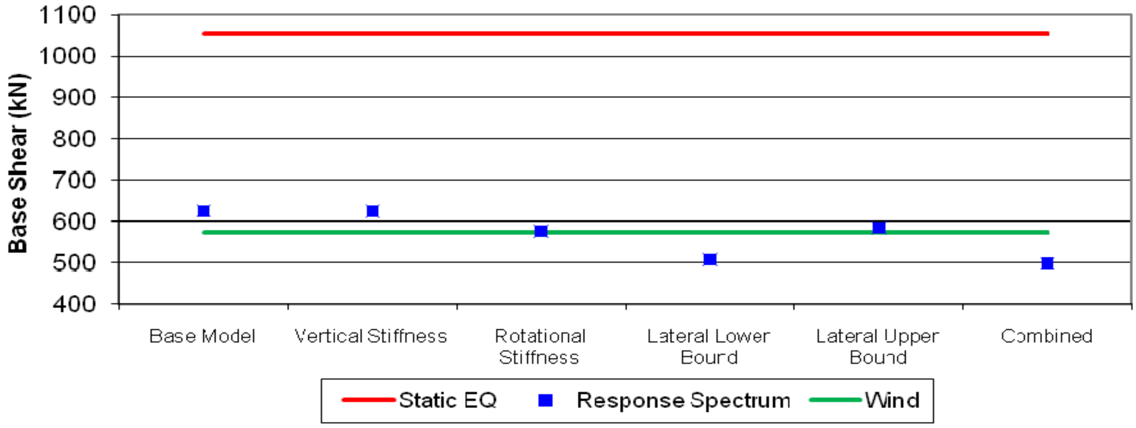

Figure 10: Foundation Stiffness Effect on Design Base Shear – Y Translation

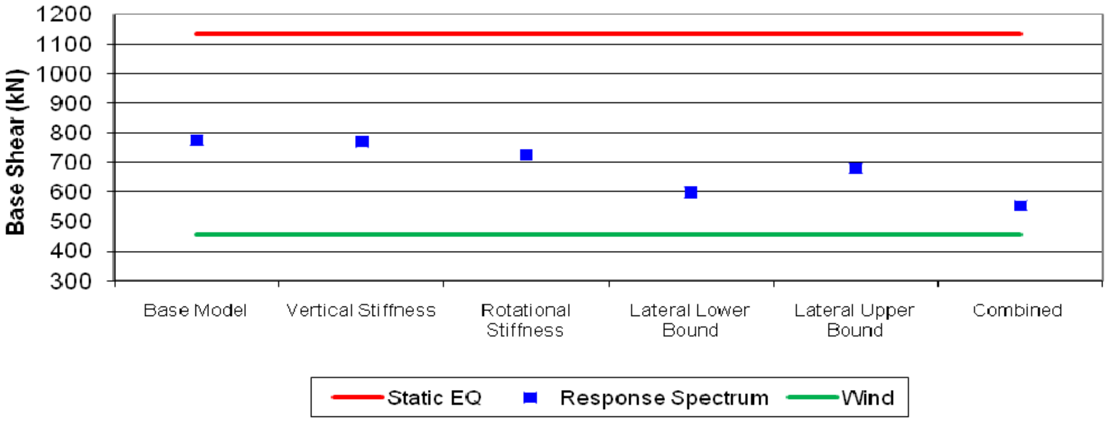

Figure 11: Foundation Stiffness Effect on Design Base Shear – X Translation

The figures clearly show that an elastic response spectrum analysis using modal superposition methods can yield a refined prediction of design earthquake actions, however an accurate mathematical model of the structure is critical if a realistic simulation of structure response is to be achieved. The response spectrum analysis results are highly dependent on the selection of structural and foundation stiffness. The pseudo-static method outlined in AS1170.4 places a minimum limit on the design actions and therefore the benefits of a flexible structure are not harnessed to their full potential.

The modal response spectrum analysis allows a more refined prediction of design actions and therefore allows a more refined design for the structure in regard to earthquake load.

3.2.4 Step 3 - First Run Analysis

The sensitivity analysis allowed the selection of appropriate section properties to be adopted for first run three dimensional analysis. The first run analysis used an elastic response spectrum analysis based on the code-specified spectrum for ground motion. The first run analysis was used to assess likely design actions that each of the elements of the structure needed to resist. The effective stiffness of the structure elements was assessed for the relevant load combinations. The assessment of the first run results showed that the columns and slabs remained uncracked at the relevant load combinations but the beam stiffness should be reduced to 35% of gross properties. The first run analysis adopted foundation stiffness assumptions derived from the sensitivity analysis.

3.2.5 Step 4 - Final Analysis

The input parameters adopted for the analysis were determined through a detailed investigation of the expected response of the structure and its individual components under the predicted earthquake loads.

The analysis has adopted a full three dimensional elastic response spectrum analysis performed in accordance with Section 7 of AS1170.4 – 2007 using the code-~~–~~specified spectrum for ground motion. The mathematical model for the building adopted appropriate material and section properties for the elements as well an accurate representation of the mass distribution of the building in line with the architectural planning. The structural model was effectively decoupled from the foundation and the effects of foundation stiffness were modelled using equivalent springs at the base of the columns. The input parameters adopted in the frame analysis are summarized as:

- Material properties in accordance with AS3600,
- Column sections assume 100% gross properties,
- Slab sections assume 100% gross properties,
- Beam sections assume 35% of gross properties.

The analysis determined the appropriate seismic load in accordance with the Australian Standard AS 1170.4 for the following parameters:

Annual probability of exceedance	P = 1:500
Site subsoil class	D_e
Hazard factor	Z = 0.05
Probability factor	$k_p = 1.0$
Structural ductility factor	$\boldsymbol{\mu} = 2.0$
Structural performance factor	$S_p = 0.77$
Earthquake design category	EDCII

The output from the final analysis was then used in the assessment of the requirement for tie beams as discussed below. The design actions from the final analysis were provided to the geotechnical engineer to determine the expected magnitude of ground movement.

3.3 STRUCTURAL ASSESSMENT OF TIE BEAM REQUIREMENT

To satisfy the Performance Requirements of the BCA and allow the footing restraint requirements of AS1170.4 to be satisfied without providing tie-beams between individual piles caps, it was necessary to demonstrate that the alternative design solution:

- Proved that pile cap displacement under earthquake loads did not induce design actions that would cause column or pile failure.
- Proved that the columns and piles were capable of carrying the gravity loads in the structure in a stable manner immediately after an earthquake.
- That the design assumptions for lateral resistance offered by the ground remained true during an earthquake.

The third item is a geotechnical question and structurally, assuming this is satisfied, it is a relatively simple exercise to determine the ability of the structure to satisfy the first two items once the predicted ground movements are provided by the geotechnical engineer. Clearly an accurate prediction of the foundation design actions is essential to determining the foundation deformation due to inertial loads. The results of the modal response spectrum analysis were provided to the geotechnical engineer to allow the inertial foundation deformations to be determined. As discussed in the following sections the inertial and kinematic deformations of the foundations were provided by the geotechnical engineer to allow the structural assessment to be completed.

The steps followed to demonstrate compliance with the above criteria are as follows:

- Step 1 – Check the column capacity for the foundation and structure deformations
- Step 2 – Check the column base shear immediately after an earthquake will not induce a sliding failure of the pile cap
- Step 3 – Check the bending moments induced in the piles as a results of the ground movements do not cause pile failure.

3.3.1 Column Capacity Check

The critical condition for column capacity is buckling and stability checks both during and immediately after an earthquake and is related to the movement of the individual pile cap and the predicted inter-storey drift of the superstructure. Therefore the ground movement that must be considered for this condition is the total pile cap movement incorporating both the inertial and kinematic ground movements.

Five columns were analysed for the predicted movements under the design earthquake event to determine if the movements are sufficient to induce instability in the structure. The five columns were selected as being representative of the columns in the multilevel car park.

Table 1: Gravity Loads Design for Representative Columns

Column Number	Dead Load (kN)	Superimposed Dead Load (kN)	Live Load (kN)
C1	5490	300	1850
C32	5340	310	1840
C50	3710	90	1740
C44	3360	90	1680
C92	3500	90	1790

Table 2: Earthquake Design Actions – Parallel to Number Grids (Y direction) for Representative Columns

Column Number	Axial Load (kN)	Shear Force Veqy (kN)	Bending Moment Meqy (kN.m)
C1	140	110	390
C32	230	130	330
C50	60	90	500
C44	10	60	380
C92	10	50	320

Table 3: Earthquake Design Actions – Parallel to Letter Grids (X direction) for Representative Columns

Column Number	Axial Load (kN)	Shear Force Veqx (kN)	Bending Moment Meqx (kN.m)
C1	190	80	210
C32	120	80	210
C50	80	60	130
C44	-	50	180
C92	-	50	150

To determine the total eccentricity due to earthquake load, D_{eq}, it is necessary to add the pile cap movement to the lateral drift of the first suspended level. The ETABS analysis was used to determine the drift of the first suspended level and for each direction the value of drift was similar. For the purposes of this exercise the largest value was adopted, i.e. lateral drift of first floor, D = 6.0 mm.

Account must be taken of the post-elastic deformations in the structure that will result during the design earthquake event. The earthquake code allows an estimate of this by multiplying the predicted elastic movements by:

$$\mu / S_p = 2/0.77 = 2.6 \qquad \textbf{(1)}$$

The total eccentricity can then be used to calculate the bending moment induced in the column and this can be compared to the design capacity of the column. The bending moment due to earthquake eccentricity can be found from the following equation:

$$M_{design} = P_{eq}\, D_{eq}\, \mu \;/\; Sp + M_{eq} \text{ (kN.m)} \qquad \textbf{(2)}$$

where P_{eq} = Earthquake axial load
M_{eq} = Bending moment from the response spectrum analysis

Adopting a predicted total ground movement (including inertia and kinematic effects) of 11mm (refer Section 4.6.3 below) and adding this to the post elastic deformations of the structure the worst predicted eccentricity for the columns is in the order of 45 mm.

The following table summarizes the axial load and corresponding bending moments for each of the five columns considered. It can be seen that the induced actions do not exceed the design capacities of the columns.

Table 4: Comparison of Design Actions Due to Combined Deformations and Design Capacities

Column Number	Earthquake Axial Load, P_{eq} (kN)	$M_{design \cdot x}$ (kN.m)	$M_{design.y}$ (kN.m)	Bending Capacity M_x (kN.m)	Bending Capacity M_y (kN.m)
C1	6535	505	684	890	2310
C32	6430	499	620	1005	2318
C50	4400	328	698	869	2144
C44	3955	360	557	711	1770
C92	4130	336	506	717	1783

3.3.2 Sliding Failure Check

It is necessary to check the shear force induced due to the predicted displacement due to the pile cap movement, first storey drift and the effects of post elastic deformation.

The shear force induced under this condition is derived from the following equation.

$$V_{base} = P_{eq}\,\mathbf{D}_{eq}\;\boldsymbol{\mu}/S_p/H \text{ (kN)} \tag{3}$$

where H = storey height (m) taken from the top of the pile cap to the top of the first suspended floor (H = 3.9 m).

The following table compares the shear induced at the pile cap immediately after the earthquake due to pile cap movement and structural deformation, to the minimum design shear force at the pile cap. It can be seen that the design base shear is greater than the induced base shear immediately after and earthquake and therefore the foundation shear capacity is not exceeded.

Table 5: Comparison of Deformation Induced Shear to Design Shear

Column Number	Shear Force After an Earthquake V_{base}(kN)	Minimum Design Shear Force V_{eq}(kN)
C1	75	80
C32	74	80
C50	50	60
C44	46	50
C92	47	50

3.3.3 Pile Capacity Check

The ground movements induce deformation in the piles which in turn induces secondary bending moments and shears that the pile must resist without failure. The additional design actions can be estimated by modelling a pile as a continuous elastic beam and subjecting this model to forced displacements to replicate the deformation profile predicted by the geotechnical analysis.

Such an analysis was performed and the resulting bending moments and shear forces were found to be negligible when compared to the structural design capacities of the piles.

3.3.4 Discussion

Fundamentally the structure responds to the ground movements, and the movements of the ground and the structure are inherently in opposite directions. Therefore the eccentricity that a column may experience and must be checked for during and after an earthquake is the combined deformations of the structure and the foundation.

As is identified in the following section the principal effect on pile cap movement is the kinematic effect and this occurs over the full depth of the soil column. The intent of the tie-beams is to limit differential movement of adjacent columns. It is difficult to see how the tie beams can limit the pile cap movement due to kinematic effects particularly when adjacent pile caps are considered to move in phase as is discussed in later sections. Based on this the tie beams can only assist by limiting the differential movement due to inertial effects. As is shown in the following sections the inertial effects only account for a small proportion of the total pile cap movement. If the tie beams cannot reduce the magnitude of the kinematic component of pile cap movement and do not add to the structural capacity of the columns or piles, then fundamentally there appears little reason for their inclusion in this instance.

It could be surmised that for structurally regular buildings in low seismic risk areas, with relatively uniform foundation conditions, there is a case for challenging the necessity for tie beams between adjacent column footings. However the process to do so requires acceptance by all stakeholders, including the owners, designers and statutory authorities.

It is clear that when considering the structural design of buildings in seismically sensitive areas that consideration of the total foundation movements is essential including kinematic and inertial effects. There is clear scope for further research into establishing when, and if, foundation ties are needed, and providing a more logical approach than that currently provided in AS1170.4.

4 GEOTECHNICAL DESIGN APPROACH

4.1 INTRODUCTION

The principle of the design approach for the alternative solution was to assess whether the passive resistance of the soil acting against the pile cap and piles can provide sufficient restraint to limit the differential movement of adjacent columns during the design earthquake event so that building collapse is prevented. This section will set out briefly an approach that was employed to assess the pile foundation performance during the design earthquake event.

4.2 METHODOLOGY OF ASSESSMENT

The pile foundations will be subjected to the following potential impact during earthquakes:

- Inertia effect – this is the effect when the piles are subjected to lateral inertia loading when the structure being supported undergoes cyclic motion during an earthquake, and is dependent on the spectral acceleration at ground level.
- Kinematic effect – this is the effect when the piles are subjected to lateral soil movement induced by the earthquake.

Both of the above effects can act together, but not necessarily in phase, to cause lateral movement, bending and shear forces on the piles.

To assess the inertia effect, equivalent static loads, as assessed by the Structural Engineer, were applied to a pile cap using Coffey's in-house program CLAP. CLAP is a development of the program DEFPIG for the analysis of axially- and laterally-loaded pile groups. It allows for simultaneous application of lateral and moment loadings in two horizontal directions and also the application of a torsional load to the pile group. It is able to handle soil profiles in which there are various layers below the pile tip, and also has a more rational data input sequence. Within the group, different pile types and different soil profiles can be considered. The analysis is non-linear and considers both geotechnical and structural failure of the piles.

For the kinematic effect, Coffey's in-house program ERLS was used to calculate the free field soil movement profile due to different earthquakes. Using the approach developed by Tabesh and Poulos (2001), the maximum computed soil movement profiles were then applied as static ground displacements to the pile, together with the inertia effects to assess the performance of the pile under the design earthquakes.

Coffey's in-house computer program ERCAP was used for the analysis of piles subjected to the earthquake-induced soil movement profiles. ERCAP uses a simplified boundary element method for the analysis of a pile/column subjected to lateral loading and/or lateral soil movement. The soil can be modelled either as an elastic continuum or as a Winkler spring model. The pile is modelled as an elastic vertical beam.

In order to assess the behaviour of the foundation under both the kinematic and inertia effects of the design earthquake, the following analysis methodology was adopted:

1) Develop design geotechnical model(s) for the site based on available ground investigation data.
2) Develop design parameters for the geotechnical model(s) for seismic site response analysis and for assessment of pile performance under kinematic and the earthquake-induced inertia effects.
3) Carry out seismic site response analysis for the three design earthquakes using ERLS and obtain the maximum lateral soil movement profile and the magnitude of surface acceleration for each earthquake event.
4) Undertake liquefaction potential assessment for the subsoil profile.
5) Assess pile performance under kinematic (i.e. subsoil movement) earthquake effects using ERCAP.
6) Assess pile performance under earthquake induced inertia (i.e. equivalent static loads at the pile head) effects using the computer program CLAP.

The following sections provide details of the above steps and a discussion of the findings.

4.3 GEOTECHNICAL MODEL

Based on the ground investigation data available for the site, the ground conditions in the vicinity of the multi-level car park comprise a sand fill layer between 2m and 3m thick, underlain by Holocene alluvial deposits up to about 20m thick. The Holocene deposits are typically described as comprising soft to firm silty, sandy clay and very loose clayey sand layers. The Holocene deposits are underlain by medium dense becoming very dense sand and gravel, which in turn is underlain by basalt rock.

Two geotechnical models were developed which were considered to represent the likely variation in ground conditions present across the site. Geotechnical Model A represents conditions where the thickness of soft to firm Holocene clay is greatest, and Geotechnical Model B represents conditions where the thickness of the very loose sand layer is greatest.

The two geotechnical models developed for use in the analysis are presented in Tables 6 and 7 below:

Table 6: Geotechnical Model A

Unit	**Description**	**Thickness (m)**	**Undrained Shear Strength, S_u (kPa)**	**SPT N Value**
1	Base course/Sand fill	2.0	-	10
2	Soft silty clay	4.5	15	-
3	Soft to firm silty clay	10.5	25	-
4	Firm to stiff clay	3.5	50	10
5	Medium dense sand	7.5	-	18
6	Basalt	Not Proved	-	-

Table 7: Geotechnical Model B

Unit	Description	Thickness (m)	Undrained Shear Strength, S_u (kPa)	SPT N Value
1	Base course/Sand fill	2.5	-	10
2	Soft silty clay	3.5	15	-
3	Very loose sand	2.0	-	3
4	Soft silty clay	8.0	15	-
5	Very loose sand	1.5	-	1
6	Very loose sand	6.5	-	4
7	Medium dense sand	4.5	-	20
8	Very dense sand	7.0	-	>100
9	Basalt	Not Proved	-	-

4.4 SEISMIC SITE RESPONSE ANALYSIS

An assessment of the seismic site response has been carried out using the computer program ERLS developed by Coffey. ERLS evaluates the response of a horizontally layered soil profile to a time-dependent acceleration at the base of the profile. Three time-acceleration histories have been considered in the ERLS analysis for detailed ground response analyses of the site for the 500 year return period ground motion, based on the findings of the Independent Seismic Hazard Assessment.

The details of the three selected acceleration time histories are given in Table 8:

Table 8: Summary of Acceleration Time Histories

Motion No.	**Earthquake**	**Station**	**Magnitude (M)**	**Distance (km)**
1	Mammoth Lakes (25 May 1980)	Long Valley Dam, Upper Left Abutment	6.3	15.5
2	Coalinga (22 July 1983)	Skunk Hollow	5.7	12.2
3	Whittier Narrows (1 October 1987)	Mt Wilson	6.0	21.1

Tables 9 and 10 set out the soil parameters adopted in the ERLS analysis for the two design geotechnical models. The small strain shear modulus (G_{max}) values for the cohesive and granular soils have been based on the following correlations:

For cohesive soils $G_{max} = 700S_u$
For granular soils $G_{max} = 5N$ (MPa)

Table 9: ERLS Parameters for Geotechnical Model A

Unit	Description	G_{max} (MPa)	Reference Cyclic Shear Strain	Damping Ratio (at very small strains)	Strain Dependent Damping Ratio (at very large strains)
1	Sand fill	50.0	3.7 x 10^{-4}	0.005	0.26
2	Soft silty clay	10.5	1.1 x 10^{-3}	0.005	0.265
3	Soft to firm silty clay	17.5	1.1 x 10^{-3}	0.005	0.265
4	Firm to stiff clay	35.0	1.1 x 10^{-3}	0.005	0.265
5	Medium dense sand	95.0	3.7 x 10^{-4}	0.005	0.26

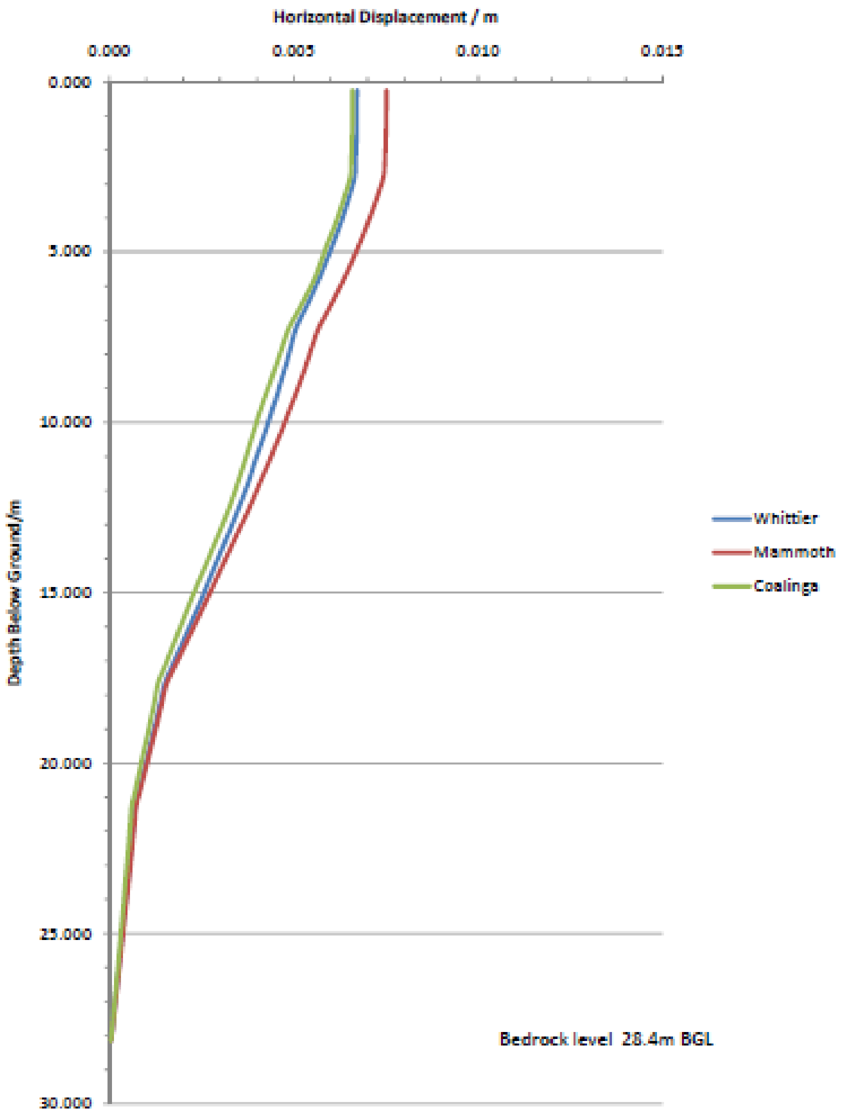

Figure 12: Free-field Soil Movement due to Design Earthquakes – Geotechnical Model A

Table 10: ERLS Parameters for Geotechnical Model B

Unit	Description	G_{max} (MPa)	Reference Cyclic Shear Strain	Damping Ratio (at very small strains)	Strain Dependent Damping Ratio (at very large strains)
1	Sand fill	50.0	3.7×10^{-4}	0.005	0.26
2	Soft silty clay	10.5	1.1×10^{-3}	0.005	0.265
3	Very loose sand	15.0	3.7×10^{-3}	0.005	0.26
4	Soft silty clay	10.5	1.1×10^{-3}	0.005	0.265
5	Very loose sand	5.0	3.7×10^{-4}	0.005	0.26
6	Very loose sand	20.0	3.7×10^{-4}	0.005	0.26
7	Medium dense sand	100.0	3.7×10^{-4}	0.005	0.26
8	Very dense sand	280.0	3.7×10^{-4}	0.005	0.26

The predicted maximum lateral soil movement profiles for geotechnical models A and B resulting from the three design earthquake events are presented in Figures 12 and 13, respectively. It can be seen from Figure 2 that for Geotechnical Model A, a maximum surface ground movement of about 7.5 mm is predicted for the Mammoth Lakes design earthquake. A slightly larger maximum surface ground movement of 8.6 mm is predicted for Geotechnical Model B for the Whittier Narrows design earthquake.

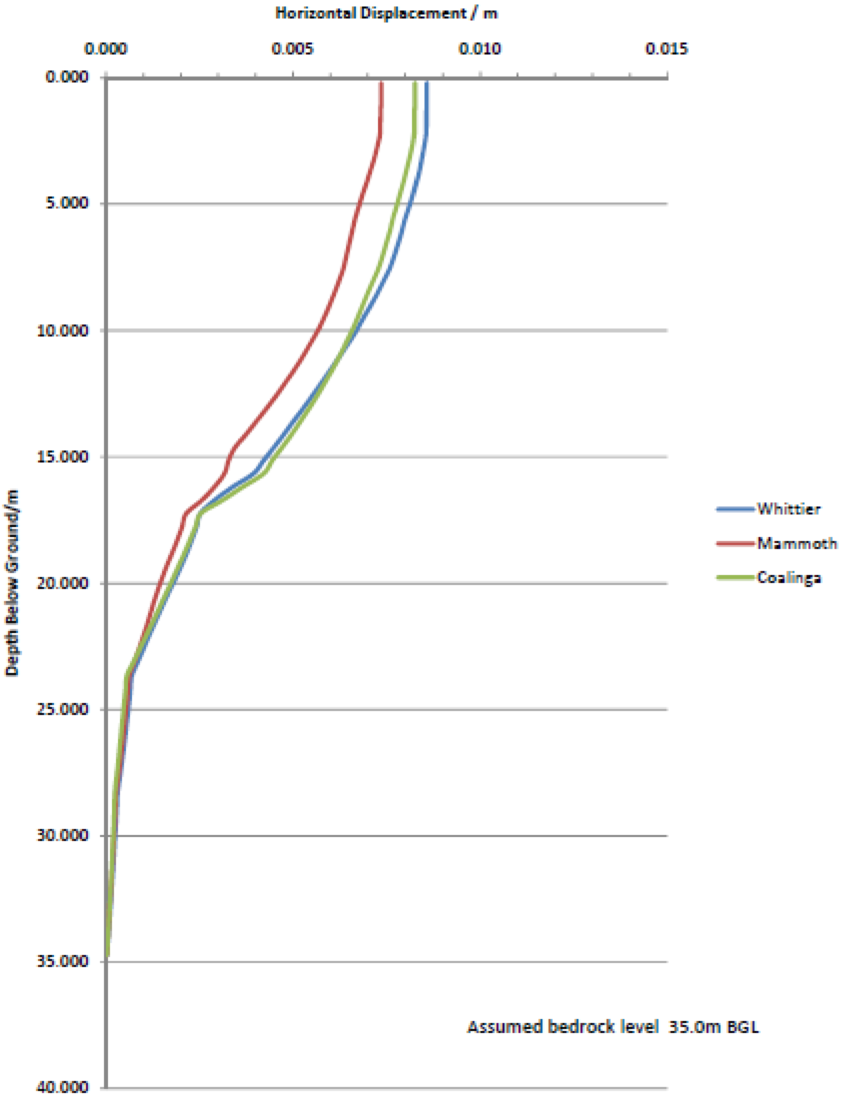

Figure 13: Free-field Soil Movement due to Design Earthquakes – Geotechnical Model B

The accelerations predicted from the ERLS analyses are summarised in Table 11 below:

Table 11: Summary of ERLS Analysis Results

Motion No.	**Geotechnical Model**	**Peak Bedrock Acceleration (g)**	**Ground Surface Acceleration (g)**	**Max. Spectral Acceleration (g)**	**Period (seconds)**
1	A	0.0396	0.057	0.2252	0.8
	B	0.0396	0.042	0.1603	0.35
2	A	0.0455	0.053	0.2003	0.9
	B	0.0455	0.041	0.1430	0.25
3	A	0.0450	0.046	0.1949	0.9
	B	0.0450	0.052	0.1360	0.25

It can be seen from Table 11 that the assessed ground surface acceleration values compare reasonably with the values given by AS1170 – 2007 for the Brisbane area, which range from between 0.05 g and 0.06 g.

4.5 POTENTIAL LIQUEFACTION ASSESSMENT

An assessment of the liquefaction potential of the ground under the design earthquakes was carried out for the site. Analyses utilising SPT data from boreholes were conducted based on the probabilistic methodology outlined by Seed *et al.* (2003) and analyses utilising CPT data were conducted based on the methodology outlined by Youd *et al.* (2001).

The analyses assumed:

- An Earthquake Moment Magnitude (M_W) of 6.3
- A maximum ground surface acceleration of 0.056 g
- The water table is at ground surface level and a soil bulk density of 20 kN/m^3
- An average shear wave velocity of 170 m/s was adopted for the upper 12 m in the Seed *et al*. (2003) analysis method.

The Seed *et al*. (2003) analysis method provides a probabilistic estimate of the likelihood of liquefaction. Moss *et al.* (2006) suggest a maximum probability threshold of 15% for design safety. The probability of liquefaction for the boreholes considered was very low.

The Youd *et al*. (2001) analysis method provides a factor of safety (FOS) against liquefaction. Robertson (2007) suggests a FOS above 1.2 provides design safety. The FOS values assessed for the CPT data considered were all greater than 1.2.

Based on the findings of this assessment, it was concluded that liquefaction of the site soils during an earthquake event is unlikely; hence any significant degradation of the foundation soils during a seismic event is also unlikely.

4.6 ASSESSMENT OF PILE RESPONSE TO EARTHQUAKE EFFECTS

4.6.1 Assessment of Inertia Effects

Coffey's in-house computer program CLAP was used to assess the pile response to the inertia effects of the design earthquakes. CLAP allows the application of loads and moments in two horizontal directions. A pile group comprising of three 350mm square precast concrete piles driven to the top of the basalt rock has been considered in the CLAP analysis. The equivalent static earthquake loads (in two directions) as provided by the Structural Engineer for Column No. C050 were applied to the pile cap, as summarised as follows:

- Axial load = 60 kN
- M_x = 500 kNm
- M_y = 130 kNm
- V_x = 50 kN
- V_y = 90 kN

Tables 12 and 13 below set out the soil parameters adopted for the CLAP analysis. To account for the stiffness of the pile cap, both the Young's modulus and lateral yield pressure values adopted for the upper 0.5 m of soil were increased. To account for the effect of the rapid and small strain loading conditions under a design earthquake, the adopted Young's modulus values were derived by applying a factor of 3 to assessed static values. The increased dynamic values are considered to be reasonable given the findings of the liquefaction assessment (i.e. significant degradation of soil stiffness is unlikely during a seismic event).

Table 12: Design Parameters for Geotechnical Model A

Unit	Description	Thickness (m)	Vertical Young's Modulus (MPa)	Lateral Young's Modulus (MPa)	Limiting Lateral Yield Pressure (MPa)
1	Base course	0.5	450	315	0.11
1	Sand fill	1.5	45	31.5	0.26
2	Soft silty clay	4.5	13.5	9.4	0.135
3	Soft to firm silty clay	10.5	22.5	15.6	0.225
4	Firm to stiff clay	3.5	45.0	31.5	0.45
5	Medium dense sand	7.5	81	56.6	1.87

Table 13: Design Parameters for Geotechnical Model B

Unit	Description	Thickness (m)	Vertical Young's Modulus (MPa)	Lateral Young's Modulus (MPa)	Limiting Lateral Yield Pressure (MPa)
1	Base course	0.5	450	315	0.11
1	Sand fill	2.0	45	31.5	0.225
2	Soft silty clay	3.5	13.5	9.4	0.135
3	Very loose sand	2.0	13.5	9.4	0.48
4	Soft silty clay	8.0	13.5	9.4	0.135
5	Very loose sand	1.5	13.5	9.4	0.985
6	Very loose sand	6.5	18.0	12.6	1.27
7	Medium dense sand	4.5	90.0	63	1.94
8	Very dense sand/gravel	7.0*	600	420	3.39

* competent rock was assumed at the depth of 35 m below ground

4.6.2 Assessment of Kinematic Effects

Coffey's in-house computer program ERCAP was used to assess the pile response to the assessed soil movements resulting from the design earthquakes. The analysis was carried out for a single 350 mm square pile and using the parameters derived for Geotechnical Models A and B as presented in Tables 12 and 13 above.

The maximum soil movement profiles assessed from the ERLS analysis for the two geotechnical models as presented in Figures 12 and 13 were applied to the pile in the ERCAP analysis. The results of the ERCAP analysis indicate that the pile head lateral deflection resulting from the kinematic ground movement is approximately the same magnitude as the soil movement (i.e. 7.5 mm and 8.6 mm for Model A and model B, respectively). This is not unexpected as the 350mm square concrete pile is of limited stiffness and therefore deflects with the surrounding ground. We note that rotational restraint from the pile cap and the structural column above the pile has been ignored.

4.6.3 Summary of Pile Response to Earthquake Effects

As discussed above, a suite of analyses was undertaken using in-house programs to assess pile foundation behaviour under both the kinematic and earthquake induced inertia effects of the design earthquakes. Our analysis approach has allowed assessment of the impacts of both soil movement and equivalent static loading in two horizontal directions on pile head deflection.

Table 14 summaries the assessed pile head displacements due to the kinematic and inertia earthquake effects for the design earthquake which predicted the maximum ground movement for the two geotechnical models. It can be seen from these results that Coffey's predicted pile head movement due to the inertia effect is consistent for both geotechnical models and is about 2.2 mm to 2.3 mm.

It can also be seen from Table 14 that the kinematic effect has a greater influence on the pile head movements, with lateral movements of between 7.5 mm and 8.6 mm predicted, giving total pile head deflections ranging from between approximately 10 mm and 11 mm.

In terms of potential differential movement between adjacent columns, the authors were of the view that the pile head movement due to the kinematic effect is likely to be in phase. Therefore it is considered the maximum differential movement between columns will largely be as a result of the inertia effects, i.e. approximately 5 mm to 6 mm if, in the worst case, the inertia induced displacements occur in opposite directions.

Table 14: Summary of Pile Response Analysis

Design Earthquake	Geotechnical Model	Pile Head Lateral Deflection (mm)		
		Inertial Effect	Kinematic Effect	Total
Mammoth Lakes	A	2.2	7.5	9.7
Whittier Narrows	B	2.3	8.6	10.9

5 CONCLUSIONS

This paper presents a case study of the seismic design for a multi-level car park constructed over soft Holocene alluvial deposits in Brisbane. The following key conclusions can be drawn from the design process:

- The assessment of the potential differential movement of adjacent columns was shown not to induce bending moments that would lead to column buckling failure and therefore will not cause collapse.
- The pile design has sufficient capacity to accommodate the inertial and kinematic movements.
- It is reasonable to conclude that the intent of the AS1170.4 to prevent collapse has been satisfied since the predicted movements do not compromise the integrity of the structure, and therefore ties between the pile caps are not required.

It could be surmised that for structurally regular buildings in low seismic risk areas, with relatively uniform foundation conditions, there is a case for challenging the necessity for tie beams between adjacent column footings. However the process to do so requires acceptance by all stakeholders.

It is clear that when considering the structural design of buildings in seismically sensitive areas that consideration of the total foundation movement, including both kinematic and inertia effects, is essential.

There is clear scope for further research into determining when and if foundation ties are needed and providing a more logical approach than that currently provided in AS1170.4.

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