INTRODUCTION

As is well known, a dramatic consequence of the Christchurch, New Zealand, earthquakes of 2010 and 2011 was the widespread liquefaction in parts of the city, which had a major effect on both residential and commercial properties. However, over a large part of the Christchurch central business district (CBD) there are layers of sandy gravel and gravelly sand near the ground surface. A number of substantial buildings are supported by shallow foundations in these gravel layers; these foundations were observed to perform well during the earthquakes and there were not large volumes of sand ejecta at the surrounding ground surface. This observation is the springboard that led to the numerical modelling discussed in this paper.

There has not been any systematic measurement of the in situ permeability of the gravels in Christchurch, and so the purpose of the present paper is to investigate the effect of permeability values expected for gravel on the cyclic pore pressure, and particularly the dissipation of pore water pressure during the cyclic loading. (The term permeability (a common soil mechanics abbreviation of permeability coefficient) as used in this paper is synonymous with saturated hydraulic conductivity.)

During cyclic undrained loading of saturated cohesionless soil there is, during each half cycle of shear strain, an incremental change in pore water pressure. Once there is a gradient of excess pore pressure, flow will occur following Darcy's law; the amount of flow will depend on the permeability of the soil. The phenomenon of liquefaction observed in sands and silty sands means that the rate of pore pressure build-up exceeds the rate of dissipation. However, there will be combinations of permeability values and drainage conditions that enable significant dissipation of pore water pressure during the cyclic loading. This is investigated in the current paper using the Cyclic1D software (Elgamal et al., 2006). Using site investigation information from the Christchurch CBD an idealised soil profile is developed. The response to sinusoidal excitation applied at the base of this column of soil/gravel is computed. As this is a relatively simple system, parametric studies into the effect of different permeability values are possible. As explained below, the Cyclic1D software incorporates a constitutive model that is capable of representing the complexity of the cyclic undrained stress–strain behaviour of saturated cohesionless materials.

Christchurch subsurface conditions

After the Christchurch earthquakes of 2010 and 2011 there was intense geotechnical investigation that has revealed the complex subsurface conditions in the city, particularly the upper 20 m or so (Canterbury Geotechnical Database, 2014). The upper layers, referred to as the Christchurch and Springston formations (Brown & Weeber, 1992), consist of sand, silt, gravel and peat layers and mixtures of these which are highly variable both laterally and vertically. The near-surface gravels, which are of interest in this study, are part of the Springton Formation (Brown & Weeber, 1992). Below the upper layer deposits, at a depth of approximately 20–25 m in the CBD, is the Riccarton Gravel, a dense gravel layer that is also the uppermost aquifer beneath the city. The age of the Riccarton Gravel is between 14 000 and 70 000 BP, whereas the ages of the Christchurch and Springton formations are less than 10 000 BP (Brown & Weeber, 1992). From the intensive street-wise site investigation data, contour maps have been prepared showing at various depths the materials encountered (Tonkin & Taylor, 2011). Fig. 1(a) has material contours between 4 and 5 m (the approximate founding level for buildings with a single-level basement) and Fig. 1(b) has contours for materials between 8 and 9 m. These material contour plots show that sandy gravel and gravelly sand are significant over a large part of the CBD. Further information about the...
Christchurch earthquake sequence and ground conditions can be found in Cubrinovski et al. (2010, 2011).

Borehole logs for two locations in Christchurch, near the location of two of the New Zealand strong motion network sites, are given in Fig. 2 and Fig. 3; Fig. 2 shows data from the Christchurch women's hospital (CHHC) site, and Fig. 3 presents data from the Christchurch botanical gardens (CBGS). The locations of these two sites are also shown in Fig. 1. Recently the two instrument sites have been thoroughly investigated with cone penetration tests (CPTs), standard penetration tests (SPTs), surface shear wave profiling, and recording of horizontal and vertical microtremor data from which the horizontal to vertical (H/V) spectral ratio is obtained (Wotherspoon et al., 2013). Shear wave velocity profiles at the site were defined using a combination of active source and passive source surface wave techniques. Layering characteristics from subsurface investigations (borehole, CPT) were used to constrain the layering of the shear wave velocity profiles during the inversion process and provide better confidence in the properties of each layer. The shear wave velocity profiles for both sites, shown in Fig. 2(e) and Fig. 3(d), show that there is little differentiation in the upper 22 m of the profile with a near-constant value for the shear wave velocity, $V_s$, with depth.

Given that there are small differences between the density of silt, sand and gravel the value of the small-strain shear modulus is dominated by the shear wave velocity (as per $G_s = \rho V_s^2$, where $G_s$ is the small strain shear modulus and $\rho$ is the density). Consequently, the upper 22 m of the profile can be approximated as having constant modulus. The period of the peak H/V spectral ratio (a method for estimating the period of a soil layer) implies, assuming a constant velocity profile above the Riccarton Gravel, a shear wave velocity somewhat less than that given by the surface wave measurements; however, this does confirm that the upper gravel layer is considerably looser than the underlying Riccarton Gravel. At both sites the boundary between the Riccarton Gravel and the overlying Springston Formations is very clear, with a jump in shear wave velocity from less than 200 m/s to 400 m/s.

The SPT values shown in Fig. 2(d) and Fig. 3(c) show that the shear wave velocity profiles during the inversion process and field measurements; however, this does confirm that the upper gravel layer is considerably looser than the underlying Riccarton Gravel. At both sites the boundary between the Riccarton Gravel and the overlying Springston Formations is very clear, with a jump in shear wave velocity from less than 200 m/s to 400 m/s.

An indication of the extent of the variability of the subsurface materials is seen by comparing the two borehole logs at the CHHC site, shown in Figs 2(b) and 2(c), which are about 25 m apart. The log for borehole 1 (BH1) shows that for the upper 10 m there is a sequence of relatively thin layers of sand and gravel with a layer of silt at the top. In contrast borehole 2 (BH2) has an upper layer of about 11 m of gravel (GW) with a thin sand layer (SP) at about 3 m. At a depth of about 22 m there is another gravel layer, the Riccarton Gravel. Looking at the borehole log for the CBGS instrument (a distance of 900 m from CHHC) in Fig. 3(b), it is apparent that this profile is similar to that for borehole 2 at the CHHC site. The borehole logs at both sites show there are sand and silt layers between about 10 m depth and the top of the Riccarton Gravel.

Generally there was little material ejected at the surface over the area of Christchurch CBD with near-surface gravel shown in Fig. 1. There were patches of ejecta found, but this is not surprising considering, for example, the log for BH1 at the site of the women's hospital in Fig. 2(b), which shows a near-surface sand or silt pocket. However, in general the lack of surface ejecta in this part of the CBD suggests buildings, particularly those on shallow foundations, were not affected by liquefaction.

An idealised subsurface profile has been used for the purposes of the paper to investigate the effect of the permeability of the near-surface gravel layer in the Christchurch CBD. The profile consists of 10 m of gravel underlain by 12 m of mixed sands and silts, overlying the Riccarton Gravel. The shear wave velocity in the upper 22 m has been assumed to be constant with depth, regardless of whether the material is gravel or sand and silt, with a value of 185 m/s, whereas the Riccarton Gravel has been assumed to have a shear wave velocity of 400 m/s.

**Permeability for Christchurch formation gravel layers**

Groundwater control is frequently needed at building sites in Christchurch and considerable volumes of water are pumped from wells. This indicates that parts, at least, of the gravel have a high permeability. Unfortunately there has not been any systematic measurement of the in situ permeability of the gravels in Christchurch.
Fig. 2. Christchurch women’s hospital (CHHC) geotechnical site investigation summary: (a) soil behaviour type index; (b) borehole BH1 log; (c) borehole BH2 log; (d) CPT $q_c$ profiles and SPT blow counts; (e) shear wave velocity profile (after Wotherspoon et al., 2013)

Fig. 3. Christchurch botanical gardens (CBGC) geotechnical site investigation summary: (a) soil behaviour type index ($I_c$); (b) borehole log; (c) CPT $q_c$ profiles and SPT blow counts; (d) shear wave velocity profile (after Wotherspoon et al., 2013)
The soil mechanics literature is light on permeability values for gravels. Holtz et al. (2011) use a value for the permeability of 0.01 m/s to separate sands from gravel; they refer to an unpublished report by Casagrande and Fadum written in 1940. Cedergren (1967) also refers to this report and presents the same values, as do Terzaghi et al. (1996). Jumikis (1962) also quotes the values from Casagrande and Fadum. Harr (1962) quotes a Russian source, which indicates that for clean gravel the coefficient of permeability is greater than 0.01 m/s. Scott (1963) gives a value of 0.01 m/s for gravel. Verruijt (2007) gives a range of values between 0.1 and 0.001 m/s. Lancellotta (2009) gives a table of values with the permeability coefficient for gravel between 1 and 0.01 m/s. Ishibashi & Hazarika (2011) state that coarse gravel has a permeability coefficient greater than 0.1 m/s, and the values for sand to fine sand are in the range 0.1–0.001 m/s. In the groundwater literature, Strack (1989) quotes an earlier publication by Verruijt with a value greater than 0.01 m/s, whereas de Marsily (1986) states that for coarse gravels the permeability coefficient is in the 0.1–0.01 m/s range. Freeze & Cherry (1979) give a range of 1–0.001 m/s. Warrick (2003) also gives 0.01 m/s as the permeability coefficient at the gravel/sand boundary. Finally, from the soil physics literature Hillel (1998) gives values between 0.1 and 0.01 m/s.

The conclusions from this review of the textbook literature are: (a) the 1940 report of Casagrande and Fadum is a frequent source of information about typical permeability values; however, those authors who do not quote Casagrande and Fadum also give values about 0.01 m/s; (b) in the absence of measured values the permeability coefficient for gravel layers could be considered within the range 0.1–0.001 m/s.

In the numerical modelling discussed below the effect of gravel permeability was investigated using values of 0.1, 0.01 and 0.001 m/s.

**Computational analysis**

To investigate the effect of the permeability of gravel on cyclic pore pressure, the Cyclic1D (Elgamal et al., 2006) finite-element program for calculating one-dimensional response of a soil column subject to lateral dynamic excitation was utilised. Non-linear soil behaviour is incorporated using incremental plasticity and thereby accounts for the effects of hysteretic soil damping. The finite-element formulation is a coupled solid–fluid approach, so enabling the generation and dissipation of excess pore water pressure.

The software has default property sets for loose, medium, medium-dense and dense materials. An important feature of the way the software is set up is that it is possible to associate any of the relative density conditions with any of the permeability values. In addition to these default materials the user can define new materials.

Hysteretic damping is implicit in the constitutive model described below. In addition, Cyclic1D has Rayleigh damping, a small amount of which is commonly used in software calculating dynamic response to assist with ensuring numerical stability; the default value of 2% at frequencies of 1 and 6 Hz was adopted for the computations discussed herein.

**Constitutive model**

It has long been known that the cyclic stress–strain behaviour of saturated sand is very complex. The general shapes of the cyclic effective stress path and the associated stress–strain curves are shown in Fig. 4. Except for very loose sand at very low confining pressure, the general behaviour is initially contractive and the effective stress path moves to the left, indicating positive excess pore pressure development. This behaviour occurs even for dense sands. However, once a threshold value of the stress ratio is reached then the behaviour switches from contractive to dilative; this happens when the effective stress path reaches the so-called phase transformation line (Ishihara et al., 1975) and is then followed by a stiffening of the soil and movement of the undrained effective stress path to the right. There are in fact two phase transformation lines, one for compressive loading and the other in extension. When the direction of loading is reversed from compression to extension the soil particle skeleton reverts to contractive behaviour and there is a reduction in effective confining pressure until the phase transformation line in extension is reached. For the first cycles there is an accumulation of excess pore water pressure, but once the phase transformation is engaged the rate of accumulation decreases.

These cyclic stress–strain phenomena occur for both loose and dense sand, the main difference is that the rate of excess pore pressure accumulation is less for dense sands. Furthermore, very similar responses are known to occur for non-plastic silt and also for gravel. Clearly these phenomena are complex and the behaviour is not elastic, consequently there is a need to use a sophisticated stress–strain–strength relationship for the soil; the model developed by Elgamal et al. (2003) is used in the Cyclic1D software. The phenomena illustrated in Fig. 4 are usually known as cyclic mobility. This transfers the attention in soil liquefaction from the idea that the effective stress in the sand is reduced to zero, to consideration of the magnitude of the cyclic shear strains in the soil. A double amplitude cyclic shear strain of 5 to 7.5% is often used as a proxy for liquefaction (Ishihara, 1996). Fig. 5 illustrates how the modelling is based on plastic yielding of the soil within a nested series of kinematic yield loci. This nesting of conical yield loci follows the concepts of Prevost (1985).

![Fig. 4. Details of the Elgamal et al. (2003) stress-strain model for sand undergoing cyclic deformation with cyclic mobility: (a) effective stress path; (b) shear stress–shear strain curve (p, mean principal effective stress; τ, shear stress; γ, shear strain; PT is the phase transformation surface) (after Elgamal et al., 2003)](image-url)
The calculations in Cyclic1D are carried out in an incremental manner such that the stiffness is evaluated for each soil element at each time step. In addition, the finite-element formulation accounts for flow of pore water, so dissipation can be modelled during the cyclic loading if the permeability is sufficiently large or if the frequency of loading is sufficiently small. Further details of the constitutive model used in the Cyclic1D software are given by Yang & Elgamal (2002) and Yang et al. (2003). The performance of the model has been validated using liquefaction case histories and centrifuge test results. Default values for material properties in various density states are given in Tables 1 and 2. Parameters which control the rate of development of cyclic excess pore water pressure are explained by Yang & Elgamal (2002) and Yang et al. (2003) and in the Cyclic1D manual where default values are given. There is also the facility in the software for user-defined materials.

MODELLING ASSUMPTIONS
Cyclic1D calculates the lateral response of a vertical column of soil divided into finite elements.

(a) Boundary conditions (Fig. 6). The deformation mode is shearing in which lateral and vertical displacements of the nodes are possible during the vertical propagation of shear waves. Lateral boundary normal stresses are generated as a consequence of the constraint that the widths of the elements do not change. There is no drainage through the lateral element boundaries, consequently any movement of water is in the vertical direction. The column of elements is underlain by an elastic half space. There is no drainage through the bottom boundary.

(b) Initial conditions. The soil is saturated (below the water table). The initial in situ vertical effective stress profile is estimated from the location of the water table and the density of the soil layers. The $K_0$ values for estimating the lateral effective stress are the Cyclic1D default values for the various relative density conditions.

(c) Soil properties. Investigation of the effect of soil/gravel permeability on the accumulation and dissipation of cyclically induced excess pore pressure was the motivation for the work reported. The Elgamal et al. (2003) soil model is assumed to represent the general features of the cyclic loading of saturated material satisfactorily. The permeability values do not change with the change in effective stress during the cyclic loading. Apart from values for permeability, Cyclic1D default parameter values are used for other properties.

Since the modelling using Cyclic1D is primarily intended to demonstrate the relation between build-up of excess pore water pressure and the dissipation rate depending on the permeability of the gravel, a sinusoidal input motion of duration 10 s is applied at the base of the soil layer. The appeal of the sinusoidal input is that it demonstrates clearly, for higher values of the permeability, the manner in which the cyclic excess pore pressure reaches a steady state with a balance between generation and dissipation. It was found with the denser materials, as the start of the sinusoidal input was very abrupt, a transient excess pore pressure response was induced. After the completion of the 10 s of sinusoidal input the computations were continued with zero base excitation for an additional 10 s to follow further the dissipation of excess pore water pressures.

Initial computations
Before the main computations some preliminary analyses were performed to determine: (a) the effects of the element

### Table 1. Cyclic1D default property values for cohesionless materials in various density states

<table>
<thead>
<tr>
<th>Density state</th>
<th>$V_s$: m/s</th>
<th>$\phi'$: degrees</th>
<th>Poisson ratio</th>
<th>Density: kg/m$^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>185</td>
<td>29</td>
<td>0-4</td>
<td>1700</td>
</tr>
<tr>
<td>Medium</td>
<td>205</td>
<td>31.5</td>
<td>0-4</td>
<td>1900</td>
</tr>
<tr>
<td>Medium-dense</td>
<td>225</td>
<td>35</td>
<td>0-4</td>
<td>2000</td>
</tr>
<tr>
<td>Dense</td>
<td>255</td>
<td>40</td>
<td>0-4</td>
<td>2100</td>
</tr>
</tbody>
</table>

### Table 2. Cyclic1D default permeability values for cohesionless materials in various density states

<table>
<thead>
<tr>
<th>Density state</th>
<th>Silt: m/s</th>
<th>Sand: m/s</th>
<th>Gravel: m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>$1\times10^{-7}$</td>
<td>$6.6\times10^{-5}$</td>
<td>$1\times10^{-2}$</td>
</tr>
<tr>
<td>Medium</td>
<td>$1\times10^{-7}$</td>
<td>$6.6\times10^{-5}$</td>
<td>$1\times10^{-2}$</td>
</tr>
<tr>
<td>Medium-dense</td>
<td>$1\times10^{-7}$</td>
<td>$6.6\times10^{-5}$</td>
<td>$1\times10^{-2}$</td>
</tr>
<tr>
<td>Dense</td>
<td>$1\times10^{-7}$</td>
<td>$6.6\times10^{-5}$</td>
<td>$1\times10^{-2}$</td>
</tr>
</tbody>
</table>

Fig. 5. Nested conical kinematic hardening yield loci used in Cyclic1D (after Parra, 1996)

Fig. 6. Finite-element mesh and boundary conditions for the Cyclic1D modelling
size, that is, the fineness of the element subdivision; (b) frequency of the input excitation; (c) amplitude of the sinusoidal input; (d) the manner in which the cyclic pore water pressures are affected by the density of the granular medium; and (e) the contrast between the properties at the base of the layer and of the underlying elastic half space. For (a)–(d) the sinusoidal input was applied at the base of a 10 m thick layer of saturated material. The underlying layer was assumed to be elastic. As explained above, the default value of 2% Rayleigh damping was used.

(a) Element subdivision. Computations with the 10 m column of material divided into 10, 20, 40 and 80 elements did not reveal any particular sensitivity to the fineness of the element subdivision. Consequently all the remaining computations were done with elements 0·5 m thick.

(b) Sinusoidal input frequency. A range of sinusoidal excitations at frequencies ranging between 0·2 and 3·0 Hz were applied to the base of the soil profile with an amplitude of 0·2g. The acceleration response spectra for the top of the profile were not particularly sensitive to the frequency of the input, having a mild peak at a frequency of 1 Hz and an even milder one at a frequency of 2·2 Hz. On the basis of these computations the input for all the calculations discussed below was at a frequency of 1 Hz.

(c) Amplitude of the sinusoidal input. A further set of calculations was done for a medium-dense profile with gravel permeability (0·01 m/s) by scaling the amplitude of the sinusoidal input excitation. Above in (a) and (b) the amplitude was 0·2g, but for the results discussed under this heading scaling factors from 0·05 to greater than 3 were applied to the amplitude of the sinusoidal input (an input acceleration amplitude of 0·2g corresponds to a scaling factor of 1·0). The results of these computations are shown in Fig. 7 with the scaling factor on the vertical axis and the peak lateral displacement at the ground surface on the horizontal axis (normalised with respect to the peak ground surface displacement for elastic response of the layer at an excitation amplitude of 0·2g). There are two lines, one for the 10 m thick layer saturated to the ground surface and the other with the water table at a depth of 2 m. Also shown in the diagram is a line at the left-hand side indicating the displacements obtained by setting Cyclic1D to calculate the elastic response of the soil layer when the water table is at the ground surface. It is clear that for both soil profiles, one with the water table at the surface and the other at a depth of 2 m, that the response of the system is highly non-linear with respect to increasing excitation. For both profiles the displacement at an input excitation of 0·2g (scaling factor 1·0) is well beyond the elastic response.

An important conclusion from Fig. 7 is the significance of the position of the water table. The largest scaling factors for the two profiles are 3·6 when the water table is at 2 m, and 2·4 when the water table is at the surface. For the surface water table case it was found that eventually there was localised shear strain within the column so the lateral displacement profile was no longer continuous. This did not occur when the water table was at 2 m depth.

(d) Effect of density on pore pressure generation and dissipation. Fig. 8 shows the outputs from the constitutive model when ten cycles of sinusoidal input of amplitude 0·2g are applied at the base of 10 m thick layer of gravel with a permeability of 0·01 m/s and the water table at the surface. There are four sets of data, one for each of the four default density conditions in Cyclic1D, using the soil property values given in Table 1. At a depth of 4·75 m the excess pore pressure history and the shear stress–shear strain history, as well as the shear stress, are plotted against the effective confining pressure. These show that only the loose and medium conditions produce significant cyclic pore water pressure but the permeability is such that the condition of zero vertical effective stress is not reached, with pore pressures dissipating very soon after the completion of the excitation. For the medium-dense and dense conditions the magnitudes of the excess pore pressures are quite small.

(e) Properties of the underlying medium. Using a two-layer 22 m deep soil profile (similar to that discussed below) the effect of the properties of the underlying material were investigated. The sinusoidal input was applied at a depth of 22 m and the properties of the underlying medium were varied from rigid to the same elastic modulus as the sand/silt layer directly above. This variation was found to have negligible effect on the generation and dissipation of excess pore water pressure in the near-surface gravel layer. Such a conclusion is quite different to the understanding derived from the behaviour of elastic layers, from which it is known that the amplification of the motion in the overlying layer is controlled by the contrast in elastic moduli between the two layers. However, as noted when discussing the results plotted in Fig. 7, the response of a 10 m thick sand and gravel profile is well beyond elastic behaviour when the input amplitude is at 0·2g.
The permeability of the lower 12 m is 6.5 m/s. For the three plots in the left-hand column the permeability of the upper layer was set to 0.1 m/s, for the middle row 0.01 m/s, and for the bottom row 0.001 m/s. In each plot for a few metres beneath the bottom of the gravel are liquefaction (zero vertical effective stress) through nearly the whole 12 m thickness. This result shows how the gravel permeability, even at a value as low as 0.001 m/s, controls the cyclic pore pressure response.

The permeability of the underlying layer has a major effect on the distribution of the excess pore pressure. When the underlying layer has permeability associated with silt, plots (d), (e) and (f) in Fig. 9, there is no dissipation of excess pore pressure during the 10 s following the cessation of the sinusoidal excitation and these pressures are indicative of liquefaction (zero vertical effective stress) through nearly the whole 12 m thickness. Note that in plots (c) and (f) of Fig. 9, the vertical effective stress in the lower 12 m is reduced to zero during the shaking; however, for the sand some dissipation has occurred in the 10 s following the cessation of the sinusoidal excitation, whereas for the silt there has been no dissipation. Also observe that there is an extremely large hydraulic gradient at the boundary between the gravel and the silt. In the case of plots (a), (b) and (c) in Fig. 9, where the permeability of the underlying layer has a value associated with sand, it is clear that the excess pore pressures for a few metres beneath the bottom of the gravel are controlled by the gravel above and so liquefaction is not initiated there. Also there is some dissipation of the excess pore pressure during the 10 s following the cessation of sinusoidal excitation.
sinusoidal excitation and the hydraulic gradient at the interface between the sand and gravel is not as severe as that in plots (d), (e) and (f) of Fig. 9.

The information in Fig. 9, then, is the main output for this paper and confirms that the permeability of the gravel layer controls the excess pore pressure. It also confirms that the rise in excess pore pressure in the loose gravel is dissipated concurrently with the build-up. In other words the excess pore pressures are never large enough to reduce the effective stresses towards zero; this is suggested as part of the explanation, at least, for the apparent good performance of the near-surface gravel layers in the Christchurch earthquake sequences. Perhaps it is appropriate to repeat at this point that the full 22 m thickness of the column has property values set for a loose material. The only difference between the upper 10 m and the lower 12 m is in the permeability.

Further insight into the behaviour of the gravel and sand/silt layers is presented in Fig. 10, where the excess pore pressure history is given at the centres of the gravel layer and the underlying sand layer; the permeability values are associated with Figs 9(b) and 9(e). Figs 10(a) and 10(c) give the time history of the excess pore pressure in the centre of the overlying gravel layer. In Fig. 10(c) the excess pore pressure soon approaches zero, but for Fig. 10(a) the excess pore pressure continues to have a small value right to the end of the 20 s computational period. The explanation is that the permeability of the underlying sand is such that the excess pore pressures near the interface are being reduced by drainage into the gravel layer with the consequent increase in pore pressure seen in Fig. 10(a).

For the computations discussed above it has been assumed that within each of the two layers the permeability is homogeneous. Investigation data around Christchurch has

Fig. 9. Excess pore pressure profiles and initial vertical effective stress profiles for loose gravel overlying loose material with lower permeability. The permeability values (k) for the upper layer cover the range for gravel from 0·1 (a) and (d), 0·01 (b) and (e) and 0·001 m/s (c) and (f). Property values for parameters other than permeability do not change.

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shown how intensely variable are the soil deposits and so the assumption of homogeneity can be questioned. Nevertheless, the fact remains that evidence of liquefaction-associated damage of the gravel layers and poor foundation performance was not observed over the part of Christchurch with near-surface gravel layers of significant thickness. Thus a reasonable explanation for the good performance of these materials is that cyclic pore pressures generated were dissipated during the course of the shaking; in other words, the average permeability of the predominantly gravel layers must have been greater than about 0·001 m/s.

CONCLUSIONS
The following conclusions were reached on the basis of the computations carried out using the Cyclic1D site response software applied to an idealised two-layer profile, based on those found in part of the Christchurch CBD, consisting of gravel overlying sand/silt.

- If the permeability values for the upper 10 m gravel layer are greater than about 0·001 m/s, any cyclically induced pore water pressures in this layer would be dissipated during the course of the cyclic excitation or soon after. A permeability value of 0·001 m/s is at the lower limit of the values for gravel obtained from the literature review.
- A permeability value in the range associated with gravel is proposed as the explanation for the apparent good performance of the near-surface gravels over parts of the Christchurch CBD during the 2010–2011 earthquake sequence, despite the fact that, on the basis of surface-wave-derived shear wave velocities, these gravels are in a loose condition so might have been expected to liquefy.
- During the cyclic loading, high excess pore pressures are generated in the lower 12 m of the soil profile consisting of sand and silt. These excess pore pressures do not dissipate during the cyclic excitation or shortly afterwards.
- Given the complexity of the soil profiles in Christchurch the numerical modelling reported herein is extremely idealised. Nevertheless, it is the authors’ view that the results reported highlight the manner in which the cyclic accumulation of excess pore water pressure and its subsequent dissipation is controlled by the permeability of the medium. For sand and silt layers the permeability is such that there is negligible dissipation during the course of the 10 s excitation, but with a larger permeability associated with gravel the rate of dissipation is able to match the rate of generation of excess pore water pressure.

The work discussed in this paper was suggested by observations of shallow foundation behaviour for multi-storey buildings in the Christchurch CBD. However, the understanding gained is of wider application in that it demonstrates that permeability values in the range associated with gravel suppress the accumulation of excess pore pressure during cyclic loading of saturated gravel.

ACKNOWLEDGEMENTS
The work reported herein would not have been achieved without the Cyclic1D software. The authors are grateful to Elgamal, Yang and Lu for encoding their constitutive model for cyclic mobility within a finite-element framework incorporating coupling between the soil skeleton and the

Fig. 10. Excess pore water pressure history at depths of 5 m (centre of the upper layer) and 16 m (centre of the lower layer). (a) and (b) show gravel overlying sand; (c) and (d) show gravel overlying silt.
pore water and for the public domain availability of Cyclic1D.

REFERENCES


