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Forced vibration testing of in situ bridge span

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ABSTRACT: An isolated span of the decommissioned SH 20 Puhinui Stream Bridge in Manakau, New Zealand was subjected to forced vibration testing in order to determine the dynamic response of the span. The bridge span was four traffic lanes wide, with seven precast concrete columns per pier and supported on precast concrete pile foundations. The span was excited along the longitudinal axis using a large eccentric mass shaker, and a benchmark system identification of the unaltered state was carried out using a MATLAB based GUI system identification toolbox (SIT) developed at the University of Auckland. Soil was then removed from around the base of selected columns with a new system identification performed after the alteration to capture the change in mode shapes and natural periods. Forced vibration testing of the bridge was able to capture a 5% increase in natural period for both modes as well as a noticeable reduction in the torsional component of both mode shapes.

1 INTRODUCTION

In recent years the importance of properly modelling soil-structure interaction has been widely investigated (Kappos and Sextos 2009; Kotsoglou and Pantazopoulou 2009; Ülker-Kaustell et al. 2010). The topic is of particular importance because if ignored, a structure can be subjected to different levels of force and displacement during a large earthquake than if a fixed based model was assumed in design (Trifunac et al. 2010).

Due to the complicated nature of soil-structure interaction, one of the inherent difficulties when modelling this effect is verifying the validity of the model. While laboratory studies can provide some insight as to how well the model describes the physical behaviour (Anastasopoulos et al. 2010), ideally testing would be carried out on full scale specimens in the conditions they are most likely to experience while in service. Forced vibration testing of in situ structures allows for this type of verification. Forced vibration testing has been used to determine dynamic characteristics of large civil structures for many decades, and is a proven method for the determination of eigenfrequencies and mode shapes (Moss et al. 1982; Samman and Biswas 1994; Halling et al. 2004; Bolton et al. 2005). This testing method was used to determine the in situ dynamic characteristics of a simple concrete bridge span in Manakau, New Zealand. The longitudinal dynamic response of the bridge span was captured and was analysed using a MATLAB based GUI system identification toolbox (SIT) developed at the University of Auckland. The captured response was subjected to a suite of system identification algorithms and the first translational and torsional modes identified. Following the benchmark system identification, soil was removed from around the base of two columns in one pier and the change in dynamic characteristics was captured.

The change in response between the two states will be used in conjunction with snapback (free vibration) testing of three columns in the southern pier to develop a series of finite element models. These models aim to accurately capture the response of bridge span and column group while accounting for inertial soil-structure interaction. This future work is briefly outlined but because of restrictions on length, only the results from the forced vibration system identifications are presented here.

2 BACKGROUND

2.1 Bridge Description

Due to the rather complicated nature of soil-structure interaction modelling, it was advisable to test a relatively simple bridge for system identification. This allowed the majority of the modelling effort to be focused on the soil-structure interaction. The Puhinui Stream Bridge in Manakau, New Zealand fit this criterion. The bridge was scheduled for demolition as a result of traffic being diverted to a new bypass which avoided several city street intersections. The bridge formerly carried four lanes of traffic on SH20 approximately 5km west of the Manakau City centre. It was constructed in 1986 with a four span superstructure consisting of eighteen 10 m precast single hollow core concrete beams per span. The beams were seated on elastomeric pads and diaphragm action between them was provided by a 100 mm thick concrete slab. The superstructure was founded on piers consisting of seven 450 mm wide octagonal concrete piles driven into silty clay deposited on top of medium-dense to dense sand. The piers were skewed at 30° and had a cross fall of 6%. Also, erosion from Puhinui Stream caused a variation in grade of up to half a metre at the columns of the southern pier. The original bridge as it existed in 2007 is shown by the outlined region in Figure 1a. While the bridge sustained no damage prior to demolition, a temporary bridge was built adjacent to the original for redirection of traffic over both bridges. The redirection of traffic allowed the demolition of the northernmost span of the original bridge to make way for the embankment of the new bypass. The position of the temporary bridge is shown by the additional outlined regions in Figure 1b and c. The Puhinui Stream was also diverted to the southern approach to make room for the bypass embankment.

(a) (b) (c)

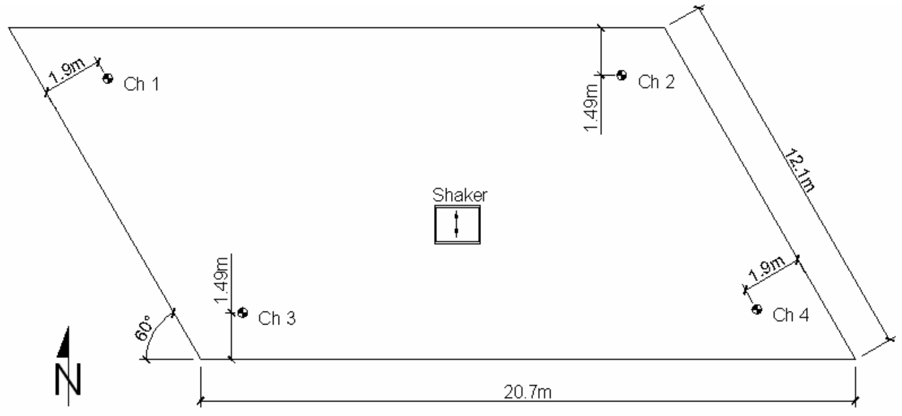
Figure 1: Changes to Puhinui Stream Bridge site during SH20 bypass is constructed (a) September 5, 2007, (b) April 4, 2009, (c) April 7, 2010

2.2 Experimental Setup

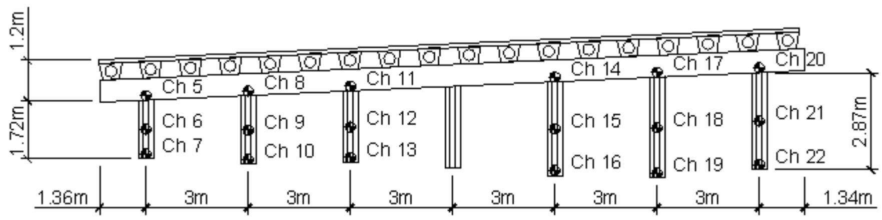
Because of the various modifications performed on the site to accommodate the new bypass, i.e. the addition of the temporary bridge, the removal of the northernmost span, and the diversion of the stream to the southern approach, the bridge system was too complicated to test unmodified and could no longer be considered representative of New Zealand bridge stock. Therefore to simplify the response of the superstructure, one span of the original bridge was isolated from the rest of the bridge during demolition.

In order to fit within the demolition schedule, the bridge span was only available for testing during one night. Therefore, in an effort to gain as much useful information as possible in such a short time span, the bridge was only excited in the longitudinal axis of the original bridge, shown by the direction of the arrow on the shaker in Figure 2a. It was reasoned that because the aspect ratio of the bridge was approximately 1:2 in this direction, the response would be more flexible than in the transverse direction and therefore easier to detect.

Once the span was isolated, it was instrumented with twenty-two accelerometers, shown by Ch 1 to Ch 22 in Figure 2. These were fixed to each corner of the deck and at three locations on six of the seven columns in the southern pier. Each instrumented column had one accelerometer 200 mm above the ground, one at mid-height, and one on the pile cap just above the column. This configuration was used in order to pick up any torsional movement of the deck, determine the level of fixity at the column bases, and capture the maximum acceleration at each column. Figure 2a and 2b give the sensor locations with ground level of the south-westernmost pile as the origin (beneath Ch 7).



(a)



(b)

Figure 2: Bridge span dimensions and sensor locations on (a) deck and (b) southern pier.

Once instrumented, the isolated bridge span (Figure 3a) was subjected to a series of frequency sweeps from an eccentric mass shaker (Figure 3b) with a maximum horizontal output of 30 kN. Three sweeps ranging from 0-4.5 Hz at 0.2 Hz steps were performed in an effort to identify the first translational and torsional modes. Once the dynamic properties of the unaltered span were determined, 450 mm of soil were removed from the two westernmost piles at channels 7 and 10. The shaking was repeated and the difference in response was determined. Future modelling efforts will use this difference to account for the contribution of the removed soil layer.

(a)

(b)

Figure 3: Isolated span (a) western elevation and (b) shaker location on deck.

3 DATA & RESULTS

3.1 Analysis Methods

The forced vibration data collected was analysed using a MATLAB based system identification toolbox (SIT) developed at the University of Auckland (Beskhyroun 2011). Twenty-two channels were collected at a sample rate of 256 Hz for each test. To reduce the effects of noise and to allow for reasonable computation times, the data set was filtered with a lowpass filter of 5 Hz and decimated to the second order.

The data set for each test was then subjected to five different system identification algorithms which were used to find natural frequencies and mode shapes. Three of the five algorithms were frequency domain based and included peak picking (PP), frequency domain decomposition (FDD) (Brincker et al. 2001), and enhanced frequency domain decomposition (EFDD). A window size of 2048 was used for these methods as it was found to provide the best resolution, while reducing inaccuracies created by zero padding.

The two time domain based algorithms applied were both variations of stochastic subspace identification (SSI) (Katayama 2005). In both methods the algorithm was run fifty times starting with a Hankel Matrix of 30 and system order of 100 which reduced by two with each iteration until the final iteration was run with a system order of two. Stable poles identified in each of these iterations were compared by one of two methods. In the first variation of SSI, the stable poles identified around the singular values generated from the singular value decomposition (SVD), were compared. If two consecutive poles within ±0.25 Hz of the singular value had frequencies within 5% and a modal assurance criteria (MAC) value greater than 0.90 both poles were kept and averaged. If both poles did not meet these criteria the first pole was discarded and the second pole was compared to the subsequent one. This series of comparisons was continued until all stable poles in the frequency range had been compared and averaged. The resulting mode shape and natural frequency are the combination of several stable poles and therefore provided a robust method of system identification.

While the first method used singular values to identify stable poles, the second variation of SSI breaks up the entire frequency range tested into 0.5 Hz bands. Stable poles are compared within each band and averaged using the same method as the previous SSI variation. Those bands with the most stable poles are considered to contain true modes and are then used to compare to the other algorithms used.

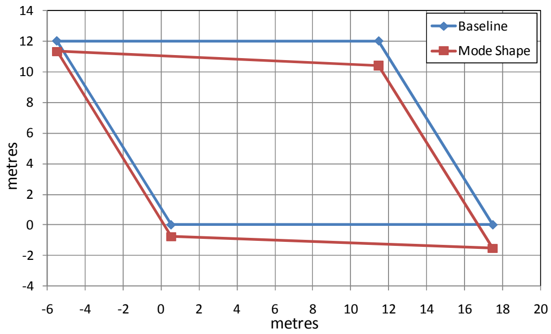
The SIT was used to calculate four modes for each system identification method in order to correctly identify the first translational and torsional modes. Modes were differentiated between true structural response and false noise modes using a two step process. First power spectral densities (PSD) were calculated for several channels and resonant frequencies were identified. A visual inspection of the generated mode shapes was then performed. If the mode shape did not include impossibilities, such as the rigid deck moving in two different directions simultaneously, and the corresponding eigenfrequencies were close to the resonant frequencies identified in the PSD, the mode was considered to be a true mode. Once the true modes were established for a given system identification method, MAC values and differences in identified frequencies were compared between the three sweeps to determine repeatability. Modes shapes were accepted if they had a MAC value of 0.90 or higher and the identified eigenfrequencies were within 15%. Using these criteria, 90% of mode shapes generated by the system identification algorithms were accepted. These mode shapes were then averaged and compared to the average mode shapes generated by the other methods. Finally, these were averaged to generate a confident mode shapes for both the unaltered and altered span.

3.2 Identified Modes

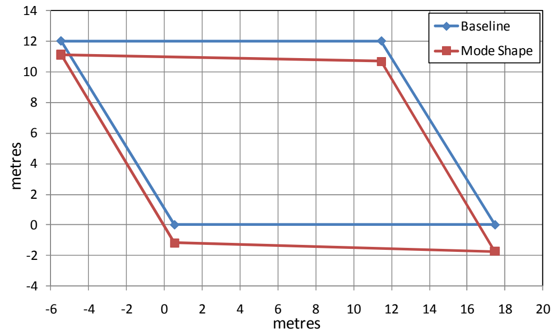
After the data gathered from the sweeps was analysed and the false modes discarded, two modes were identified for the unaltered state. The first mode had a natural period of 0.429 s and the second mode occurred at 0.252 s. Both identified modes were primarily translational with a torsional component.

This torsional component was more pronounced in mode 1 than in mode 2 as can be shown in Figures 4a and 4b in which the modal amplitudes were scaled by a factor of ten and plotted against the baseline location of the sensors. This method of plotting was used instead of double integrating the acceleration data because the mode shapes were composed of a compilation of different tests and system identification algorithms. If double integration was used, it would disregard much of the analysis used to derive the compilation mode shapes. Also it should be noted that the transverse component of the torsional movement of the deck was not captured because the accelerometers only measured accelerations along the longitudinal direction.

To investigate the modal response of the columns, the modal amplitudes of each sensor was plotted with respect to the distance the sensor was from the origin i.e. the westernmost column (Figures 4c and 4d). This plot essentially gives a plan view of the mode shape from the pier cap down in which the longitudinal modal amplitudes of each column sensor can be compared. Using this plot the torsional response of the system becomes clear, with modal amplitudes increasing as you move from the left to right side of the figure. This torsional response was expected because the easternmost column (channels 20-22, right side of the figure) was almost 1.5m taller than the westernmost column (channels 5-7, left side of figure) due to the 6% cross fall and was therefore approximately 3.5 times more flexible than the westernmost column. As the column length increases due to the cross fall, the modal amplitude of the pier cap increases approximately linearly until it is 0.27 higher at the easternmost column in mode 1 and by 0.24 in mode 2 at the westernmost column.



(a)



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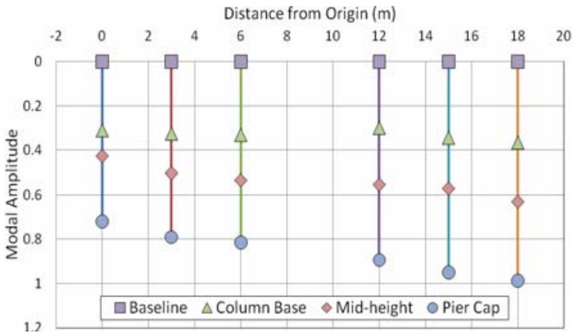

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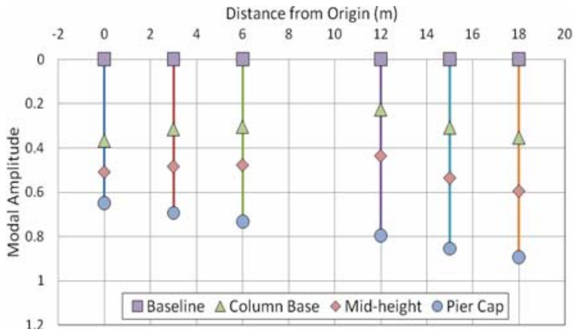

(d)

Figure 4: Unaltered span mode shapes (a) plan view of deck modal amplitude: mode 1, (b) mode 2, (c) plan view of column modal amplitude at column base, mid-height of column and at pier cap of each column: mode 1, (d) mode 2

The 30° skew of the span also introduced some interesting effects especially with the three easternmost columns. Due to the skew the two easternmost columns have no other columns in line with them in the direction of shaking. This makes both columns much more flexible than the other five columns in the same pier. Additionally due to erosion from Puhinui Stream, all but the two easternmost columns in the northern pier were 3.0-3.7 m long. The two easternmost columns had soil eroded to 3.5 m on the stream side but only to 2.5 m on the other side, making these two columns almost twice as stiff as the others in the northern pier. These two stiff columns line up with the centre column (not instrumented) and the column directly east of centre (channels 14-16) on the southern pier. Since the stiff column of the northern pier was in line with channels 14-16, the modal amplitude of the column instrumented with channels 14-16 was reduced. This reduction in stiffness is apparent when the modal amplitudes at the column bases are studied in mode 1 (Figure 4c). The modal amplitudes increase linearly until the column 12 m from the origin. There the modal amplitude of the column base is reduced by 12% from the expected linear increase. This reduction in stiffness appears to be less with increased height of measurement, as the mid-height and the pier cap modal amplitudes follow a more linear increase.

The pier cap and deck modal response in mode 2 are similar to that found in mode 1, except with a smaller amount of torsion (Figures 4a and 4d) and an overall reduction in modal amplitude (Figures 4c and 4d). This similarity does not exist at the mid-heights of the columns or at the column bases. Where in mode one the response at these levels was generally translational with a torsional component, the response has changed in mode 2 to a three node curve with antinodes at approximately 4 m and 13 m from the origin. This effect dies out at the pier cap as the inertial force from the rigid deck dominates the response as can be seen in the three dimensional plot of the bridge span deformed in the second mode from SIT (Figure 5).

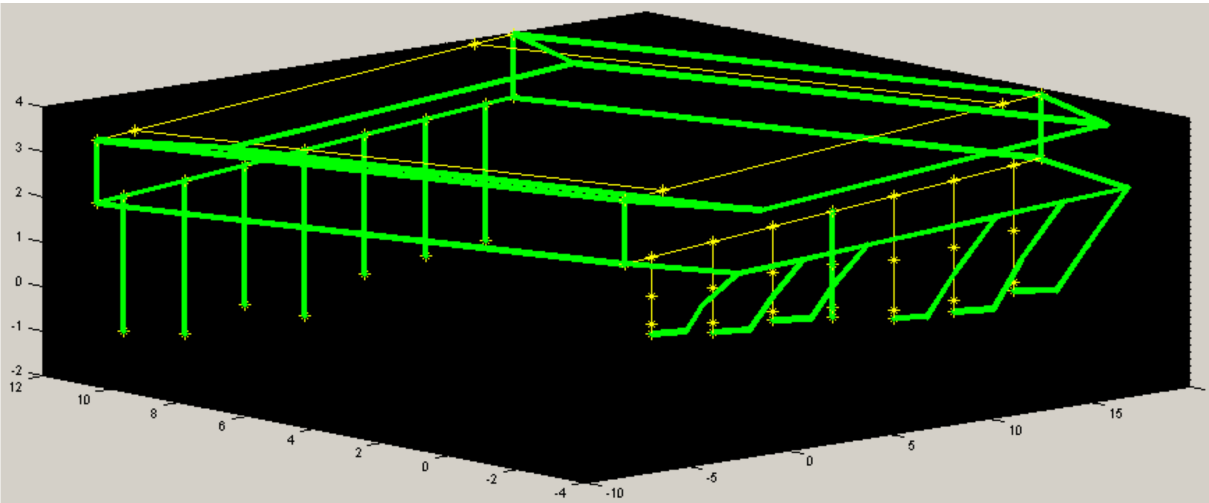

Figure 5: 3D plot of unaltered bridge span mode 2 from SIT. Thick green line denotes mode shape.

After removing 450 mm of soil from around the column bases of the two westernmost columns on the southern pier (red circles in Figure 6), there was approximately a 5% shift in natural frequencies. The first mode increased to 0.449 s and the second mode increased to 0.270 s as a result of the reduction in stiffness of the system. There was also a noticeable change in mode shape with the majority of the change occurring at this deck level where the torsional effect almost disappeared (Figures 6a). This response was expected as the removal of soil increased the effective length of the columns and moved the centre of rigidity of the system closer to the centre of mass. The reduction in torsional movement was also pronounced at the pier cap, where in mode one change in modal amplitude with respect to distance from the origin decreased by 15%. It should be noted that the two westernmost columns, those that had the soil conditions altered, had the largest changes in modal amplitudes, increasing by 9% at the pier cap (Figures 6c). This increase in modal amplitude occurred at all columns with increases showing a linear trend between the 9% at the westernmost column and 2% at the easternmost column.

The second mode of the altered span lost much of the torsional component of the response. The response changed to become dominated by the three node curve that was identified from the column base and the mid-height accelerometers in mode 2 of the unmodified span (Figure 6d). This mode also increased in flexibility at all sensors by a similar linear increase as in mode 1, with the maximum increase in modal amplitude of 10% occurring at the westernmost column and reducing to a minimum increase of 1% at the easternmost column.

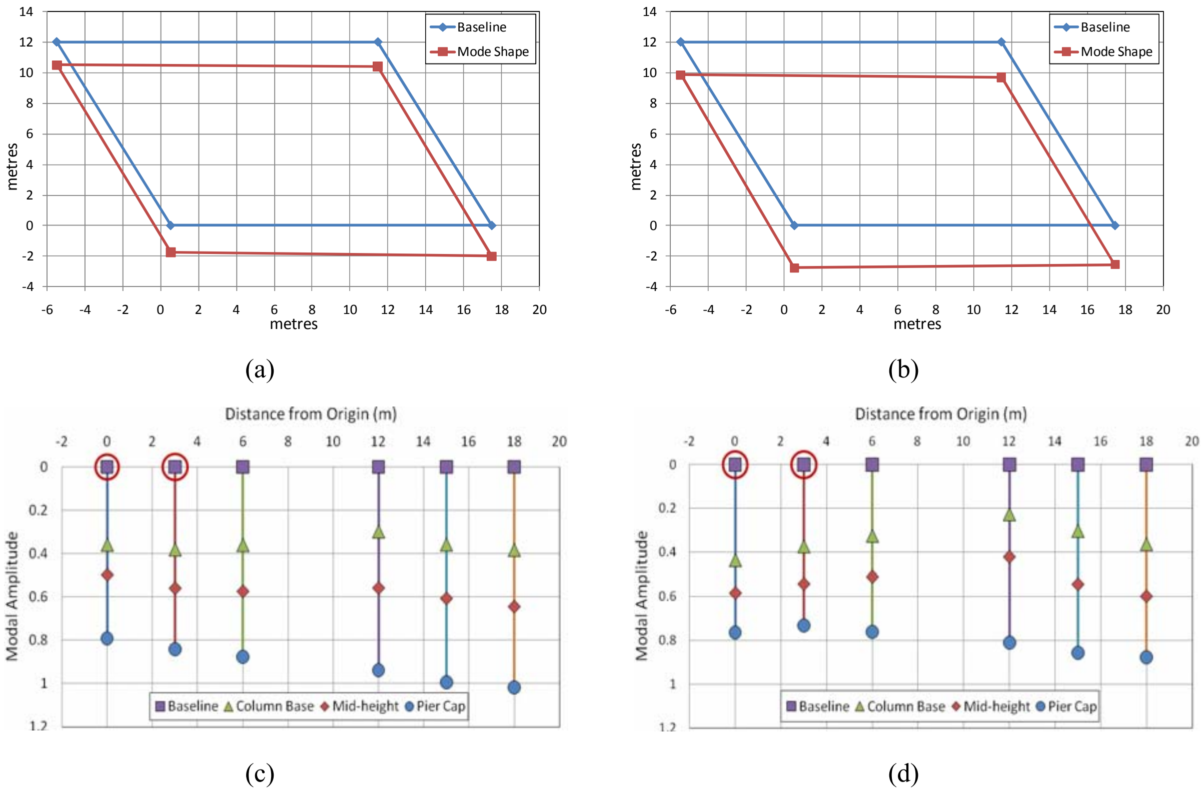

Figure 6: Altered span mode shapes (a) plan view of deck modal amplitude: mode 1, (b) mode 2, (c) plan view of column modal amplitude at column base, mid-height of column and at pier cap of each column: mode 1, (d) mode 2

4 CONCLUSIONS & FUTURE WORK

Forced vibration testing of the Puhinui Stream Bridge was able to capture the change in dynamic response caused by the removal of only 450 mm of soil from two columns. This alteration to the bridge system increased natural periods of both identified modes by 5% as well as a reduced the torsional component of both mode shapes that was present in both modes of the baseline system. The removal of soil from the base of the two columns also increased the modal amplitudes of all sensors, with the maximum increase of modal amplitude occurred at the westernmost column and decreased linearly the further a sensor was from the modified columns. The collection and analysis of this in situ data was made possible by the use of SIT and the suite of system identification algorithms it utilizes.

Following the system identification, the span was partially demolished, isolating three columns connected by the pier cap at one end of the span for testing. A series of snapback tests were performed on these three column system to determine the dynamic response and damping characteristics (not described here). The forced vibration and snapback data will be used to develop a series integrated soil and structural models for this bridge span. Both the unaltered and altered states will be modelled so that the contributions of the removed soil can to be directly quantified and the robustness of the model can be tested by using two different boundary conditions. These integrated models will be compared to simplified fixed base models and the appropriateness of these simplifications will be investigated thus providing a greater understanding of soil-structure interaction and lay the framework in providing guidelines for this modelling technique to be taken up in design practice.

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