Three-dimensional finite element modelling of a bridge structure with rocking piers under cyclic loading

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Abstract

The development of rocking in bridge structures has been identified as a valid isolation technique for structures under seismic loading. By utilizing uplift in the bridge system, ductility and strength demands can be reduced on the structural element. This limits the damage to the element, and reduces residual displacements of the structure due to the system's self-centring capability. Whilst multiple experimental studies on this isolation method have been conducted by researchers, limited focus has been placed on numerical models, specifically three-dimensional micro-models, which allow for better understanding of the actions of the bridge structure undergoing this motion. The objective of the present research is the development and validation of two three-dimensional finite element models undergoing cyclic loading, using the software package ANSYS - a conventional reinforced concrete monolithic bridge pier and a precast post-tensioned concrete bridge pier wrapped in FRP which allows uplift. The validation of these models according to existing experimental data focuses on the damage of the bridge pier under sustained loading and the corresponding concrete constitutive models utilised. Once the models have been validated, further work allows for easy identification of the benefits of FRP and rocking. The models may also be applied to future research involving improved structural systems and larger bridge structures.

Keywords: rocking, bridge piers, cyclic loading, residual displacement, finite element, post-tensioning, FRP

1. INTRODUCTION

Conventionally designed bridges rely on inelastic behaviour of the piers to dissipate energy during seismic events, which can result in significant damage and residual displacement of the structure, sometimes requiring demolition. In an effort to reduce this damage, elastic self-centring systems have been introduced, which utilise the idea of ‘rocking’ by allowing uplift to occur. In the case of bridges, this uplift may be allowed at the pier-foundation interface such that only elastic deformation occurs, and therefore minimal, if any, residual damage is incurred.

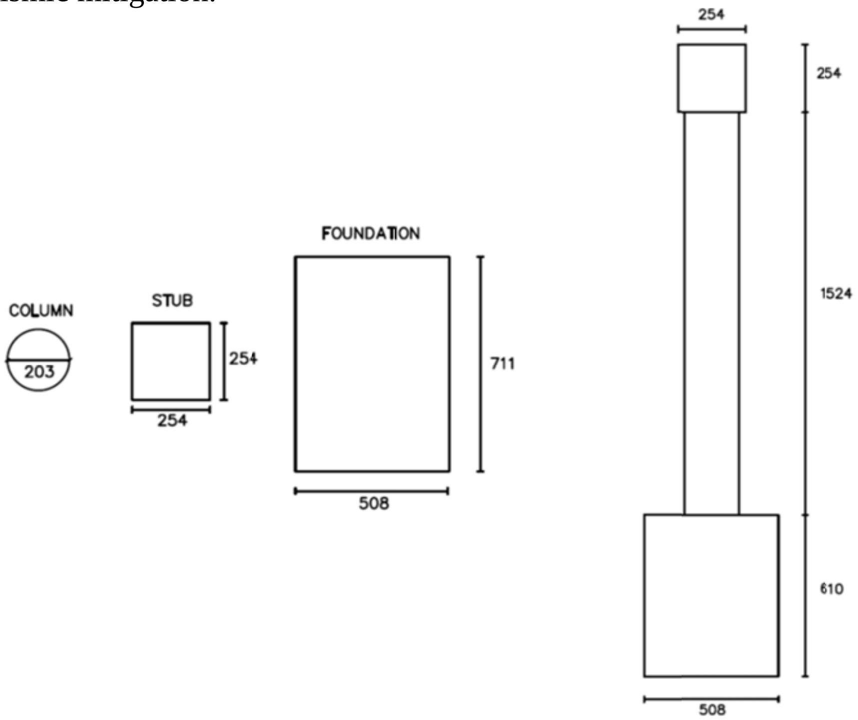
Early work on the rocking mechanism in structures was done by Housner (1963), and furthered with studies by Priestley (1991) focusing on the use of post-tensioned tendons for use in precast frames, whilst allowing a rocking mechanism to develop. The tendon assists in the structural stability and self-centring of the system, whilst the movement towards precast solutions for building systems has risen from the need for cheaper and more efficient construction methods. This ‘hybrid’ system allows for large lateral displacements to occur with little permanent structural damage or residual deformations. Rocking columns for use in bridge substructure design have utilised variations of the ‘hybrid’ solution in studies by various researchers, with experimental work focusing on single bridge pier models. Some studies have included additional energy dissipative devices, such as mild steel bars (Palermo et al. 2007), which have the potential to provide greater system stability along with increased energy dissipation capacity, and the use of externally wrapped fibre-reinforced polymer (FRP) as an alternate form of reinforcement and stability (ElGawady et al. 2010). In the computational modelling of these systems, emphasis has been placed upon simplified analytical models of the lumped plasticity and multi-spring model type, and have been summarised by Palermo et al. (2005). Few finite element (FE) studies exist, with those completed showing deficiencies in the definition of the concrete material model. Studies by Ou et al. (2007) and Dawood et al. (2012) utilised the computational software ABAQUS to develop three-dimensional (3D) FE models of single segmented piers, subjected to cyclic and monotonic loading respectively. At larger drift values the models failed to sufficiently capture strength and stiffness degradation, hence under-estimating residual damage.

This paper details the development of a 3D FE model of a single precast concrete bridge pier, utilising post-tensioning and FRP wrapping as confinement. This model is subjected to cyclic loading, and allowed to uplift and form a rocking motion. Similarly, a model of a conventional monolithic reinforced concrete column is also developed. Special attention is paid to the definition of the concrete material models utilised and results are compared to existing experimental data for validation purposes. After validating the initial two models, it is possible to develop a third model which is monolithically constructed, but introduces the use of FRP. The comparison of the three models then allows the benefits of the use of FRP, and the use of the elastic rocking system to be demonstrated clearly.

2. 3D FE MODELS

Three 3D FE models have been developed, representing a single bridge pier. The models consist of a circular column, the bridge substructure in the form of a foundation section, and the bridge superstructure in the form of loading stub. The first model is a reinforced concrete monolithic bridge column, conventionally designed such that plastic hinges form upon repeated lateral loading. The second FE model replicates the geometry of the first model, but is modelled with a precast post-tensioned system, is wrapped in FRP, and is allowed to develop uplift during the application of lateral forces. Once the two original models are validated, a third model is developed utilising monolithic construction, but introducing FRP wrapping as an additional reinforcement technique.

For validation purposes, the models utilised the dimensions and loading configuration of the experimental tests completed in Booker (2008) and published in ElGawady et al. (2010). The common dimensions of the two models are shown in Figure 1, each model consisting of a circular column 203 mm (8 in) in diameter, and 1524 mm (60 in) long. The load stub is 254 mm (10 in) square, and the foundation is 711 mm (28 in) long by 508 mm (20 in) wide by 610 mm (24 in) deep. The experimental work by the mentioned researchers was chosen on the basis of the use of FRP as a confinement mechanism, together with precast construction. These materials and techniques are relatively young in the construction industry, but both systems are becoming increasingly popular in the area of seismic mitigation.



(a) (b)

Figure 1*: General model dimensions in mm (a) cross-section dimensions (b) pier dimensions*

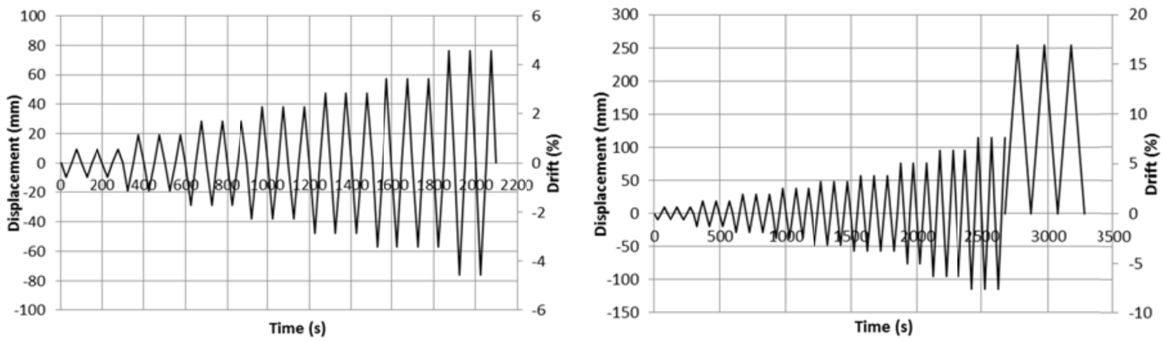
2.1 MONOLITHIC MODEL

The monolithic model consists of reinforced concrete as per the dimensions given in Figure 1. The experimental column had longitudinal reinforcement provided by six (equivalent) No.10 Grade 420 bars (No. 3 Grade 60 in inch-pounds) representing a 1.31% reinforcement ratio, and shear and confining reinforcement provided by a spiral at 102 mm (4 in) pitch of (equivalent) No. 6 Grade 280 (No. 2 Grade 40 in inch-pounds), representing a transverse reinforcement ratio of 0.31%. The concrete was tested with a compressive strength of 13.8 MPa (2 ksi) (Booker 2008).

The SOLID65 element is used for the structural concrete body of the column, which is a 3D structural solid element in ANSYS dedicated to the modelling of reinforced concrete (ANSYS Inc. 2011). It is defined with smeared reinforcement according to the reinforcement specifications given in the experimental work. The mesh size was based upon a balance between computational time and output accuracy, with a finer mesh used at the column-foundation interface where damage is concentrated. Symmetry conditions are defined, and confinement is allowed for by increasing the strength of the defined concrete according to the model of Mander et al. (1988). The foundation is heavily reinforced to prevent failure from governing the behaviour of the system.

The SOLID65 element has been used by many researchers for FE modelling of concrete, with different material models and hence varying results. The cracking and crushing capabilities of the element are defined by a failure surface based upon the William-Warnke failure criteria (ANSYS Inc. 2011), assuming linear stress-strain behaviour to either the maximum tensile or compressive stress specified. After the failure surface has been reached, the element stresses are reduced to zero, with the element effectively eliminated, and with no capacity for softening behaviour. The assumption of a linear stress-strain relationship, and the absence of softening behaviour mean that the element, utilised by itself, is inadequate to represent the nonlinear behaviour of concrete in a complex model, such as one required for a full analysis of structural performance. Several preliminary studies on the effectiveness of the SOLID65 element and its settings and real constants were performed in conjunction with the developed FE model. It was determined that the SOLID65 element, utilised with smeared reinforcement, and without the William-Warnke compressive failure surface (eliminating crushing criteria), together with an elastoplastic multilinear kinematic hardening model, resulted in the most effective constitutive model for this analysis. This finding is in keeping with previous research (Barbosa and Ribiero 1998; Kachlakev et al. 2001).

A cyclic loading cycle, as used in the tests performed by Booker (2008), was utilised in the analysis of the column, consisting of displacements which are increased in subsequent cycles up to a drift of 4.6%. This loading sequence is shown in Figure 2(a).



(a) (b)

Figure 2: *Applied cyclic loading (a) up to 4.6% drift and (b) up to 6.9% and postive cycles up to 15.4% drift*

2.2 PRECAST POST-TENSIONED MODEL

This model details a circular precast single bridge pier utilising an unbonded post-tensioned tendon and externally applied FRP, with the same geometry and mesh size as the monolithic model. The experimental column had a post-tensioned tendon consisting of a hot-rolled threaded bar of 32mm (1.25in) diameter with an ultimate strength of 1034MPa (150ksi) as the only longitudinal reinforcement present in the column. The shear and confining reinforcement was provided by the FRP, which consisted of a circular fiberglass reinforced polymer tube with a thickness of 3.175mm (0.125in). The manufacturer stated circumferential flexural modulus is $24.8E^3$ MPa ($3.6E^6$ psi), and Poisson's Ratio is 0.35 (Booker 2008).

Symmetry and confinement are defined as previously, but the theoretical approach of Mander et al. (1988) was altered slightly to account for the FRP confinement. The FRP is also physically modelled in the simulation for the purpose of providing stability and added protection to the concrete elements at the base of the column. This was found to be necessary in order to replicate the reduced damage observed at the base during the rocking motion in experimental studies. No additional confinement pressure was applied to the model by the modelling of this FRP. FRP failure and FRP-concrete interaction in the form of delamination were not considered.

The built-in concrete element (SOLID65) available in ANSYS was not used in this numerical model, as the treatment of the concrete cracking and crushing behaviour was found to be unsuitable. Several preliminary analyses using the SOLID65 element were performed, but it is thought that the lack of any further steel reinforcement (in addition to the unbonded tendon) in the structure resulted in excessive premature tensile cracking of the concrete at the base of the column which led to solution divergence. The premise behind allowing a structure to uplift and rock during lateral loading is based upon the need for damage avoidance, with previous research showing reduced structural damage during dynamic or cyclic loading. Furthermore, the use of FRP has been shown to further reduce damage at the base of the column. It was thus inferred that a simpler solid element may be used to represent the concrete in the rocking FE model. However, as less absolute element failure is expected, the compressive softening branch of a typical stress-strain curve for reinforced concrete becomes more important than the tensile cracking strength in this model. As the SOLID65 element cannot detail softening behaviour, and has already been determined to be unsuitable in the current analysis, a more stable method of reducing the strength and stiffness of the model at increasing strains is needed. The 3D structural solid element SOLID185 was thus used in this model, together with an elastoplastic multilinear isotropic hardening model. In order to gradually reduce the strength and stiffness of the elements as the analysis progressed, the ANSYS command ‘MPCHG’ was used (ANSYS Inc. 2011), which allows the material of an element to be changed during an analysis. Several different materials were defined with a reduced stress-strain curve specified for each subsequent material. Following each period of the loading cycle, the strain in each concrete element is determined, and the material of that element can then be changed accordingly, without stopping the analysis. In this way, a softening behaviour is imitated, as the strength of the elements is reduced with increase in strain. Elements with a strain greater than 0.06, which is taken as a state of crushing for confined concrete, are assigned a very low strength, which maintains stability.

The unbonded post-tensioned tendon is defined by a general metal plasticity model, with the tendon running through the centre of the column system. The tendon remains unbonded through the body of the column, but is anchored in the foundation and loading stub, providing a post-tensioning equivalent to 30% of the tendon’s ultimate capacity. 3D surface-to-surface contact was specified on both the column-foundation interface, and the column-loading stub interface, which allows for interactions between the model surfaces during the development of the rocking motion to be monitored. Rough contact requires a defined friction coefficient of 0.5 to be overcome before uplift may occur.

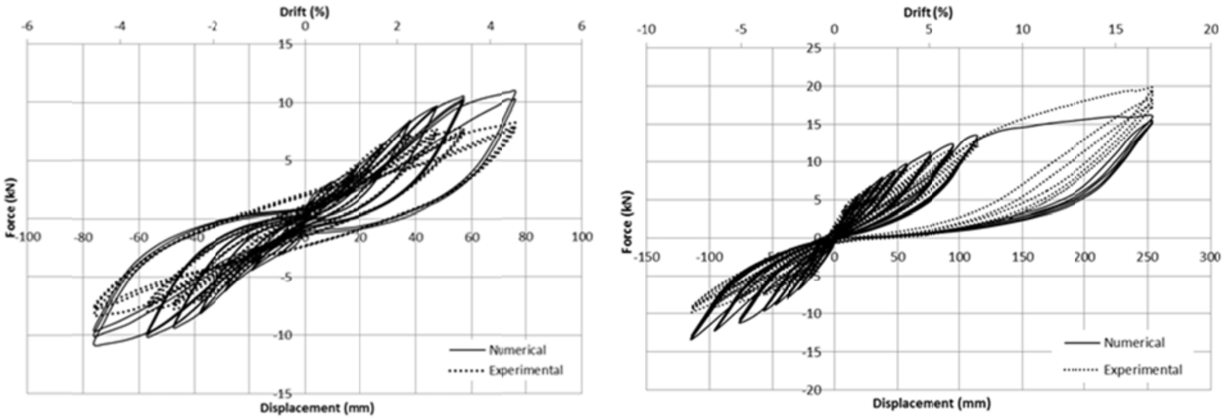
This simulation uses the full loading sequence from the tests by Booker (2008) shown in Figure 2(b). The full loading sequence cycles up to 6.9% drift, followed by a secondary loading sequence of three half cycles of positive displacement only, up to a drift of 15.4%. Results up to 4.6% drift may be extracted for comparison with the two additional models, whilst the larger drifts in the secondary loading sequence allow further insight into any permanent damage caused by the rocking behaviour.

3. MODEL RESULTS

The numerical models were validated by using the experimental results from Booker (2008) and published in ElGawady et al. (2010). The original experimental data was kindly supplied to the authors by M. ElGawady (personal communication 13 July 2013). The results are compared to the experimental data by the use of force-displacement hysteresis loops, with the results from the numerical model displayed as a solid line, and the experimental results displayed as a dotted line.

3.1 MONOLITHIC MODEL

The hysteresis loops for the monolithic column are shown in Figure 3(a). The numerical model shows an increase of the peak load by 24%, and it appears that the utilised material model slightly overestimates initial system strength and stiffness and hence underestimates the damage incurred in the structure. However, the utilised material model shows adequate strength degradation, and comparison shows a good general fit, with similar energy dissipation and residual displacements as summarised in Table 1.



(a) (b)

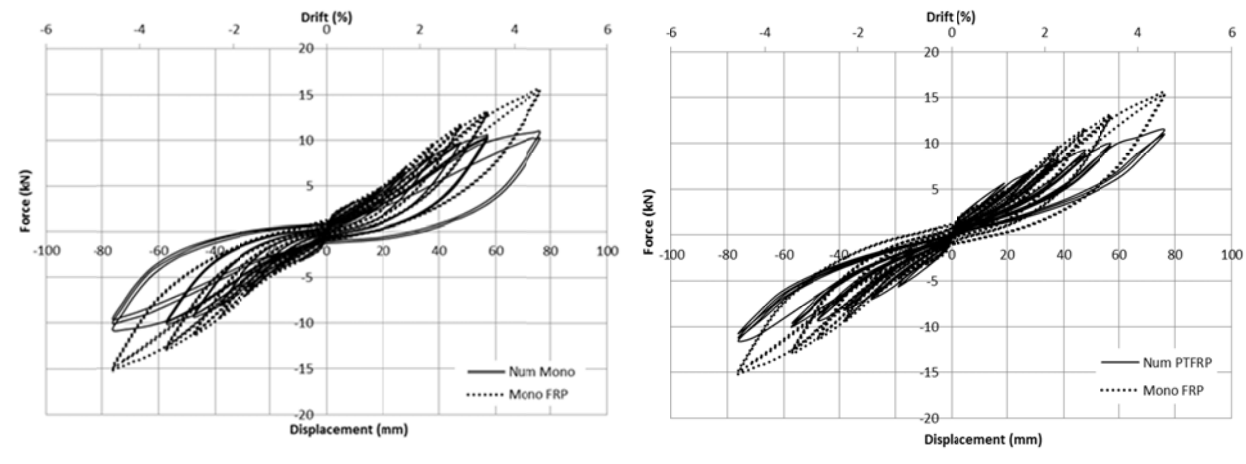
Figure 3: *Numerical and experimental results for (a) the monolithic column and (b) the PTFRP rocking column*

3.2 PRECAST POST-TENSIONED MODEL

The hysteresis loops for the post-tensioned FRP (PTFRP) model are shown in Figure 3(b). Considering the positive cycles of the primary set of loading first (the full cyclic loops up to 6.9% drift), good agreement is shown in terms of initial and post-yield stiffness, energy dissipation and residual displacement. The difference in peak loads for the primary loading sequence is 5%. The negative cycles of this loading set, however, show some anomalies in that the experimental data does not produce a symmetrical force-displacement relationship in the positive and negative cycles as would be expected. As per personal communications, Elgawady (July 2013) mentioned that there were some irregularities in the test data which were attributed to the method of construction of the sample, and appears to have caused some abnormal movement about the zero axes. As such, for the validation of the numerical model, the positive half cycles are deemed to be the best indicator of the sample performance. The results for the secondary loading cycle, which consist of the three positive half cycles up to 15.4% drift, also show good correlation between the stiffness values and the energy dissipation. The relatively large difference between peak values implies that the numerical model shows too much degradation for larger drift values. As good correlation is shown up to 6.9% drift, and stiffness and energy dissipation can be seen to be similar in the larger cycles, the discrepancy between the peak values was not deemed to be as important for the 15.4% drift cycles, and it may be concluded that the concrete model utilised gives an adequate approximation of the concrete behaviour.

3.3 BENEFITS OF FRP WRAPPING AND ROCKING BEHAVIOUR

After validation with existing experimental results, the monolithic and FRP wrapped precast post-tensioned columns may be used as a basis for further models, such as the monolithic FRP wrapped column system which has been simulated. The results of all three models allow for the benefits of both FRP wrapping and allowing column uplift and rocking under cyclic loading to be identified.
The numerical results for the primary loading set (up to 4.6% drift) of the monolithic column wrapped with FRP (Mono FRP) are shown in Figure 4(a) (dotted line) in comparison to those from the conventional monolithic column detailed in Section 3.1 (solid line). Of greatest interest is the reduction in residual displacement of the system, as summarised in Table 1, which equates to a 53% reduction and indicates significantly reduced permanent damage in the column system with the addition of the FRP wrapping. Furthermore, despite the reduced damage in the column, the hysteretic energy dissipation in the FRP wrapped column is only 5% less than that shown in the conventional monolithic column. The peak load has also increased by 30%, which is to be expected due to the FRP acting as an additional source of reinforcement. The results of the monolithic FRP column may also be compared to the FRP wrapped precast post-tensioned column (PTFRP) up to 4.6% drift, as shown in Figure 4(b). Most notably, the UBPT and rocking motion in the PTFRP column acts to decrease the residual displacement to near zero as compared to the monolithic FRP column, as summarised in Table 1. Slower strength and stiffness degradation show a further decrease in permanent damage as opposed to the monolithic FRP and monolithic columns. Figure 3(b) also shows the capacity of the PTFRP column to withstand high drifts with minimal residual damage. There is, however, a significant drop in hysteretic energy dissipation in the PTFRP model.



(a) (b)

Figure 4: *Results for the Monolithic FRP wrapped model compared to those for (a) the conventional monolithic model and (b) the PTFRP model*

A summary of the residual displacements and residual drifts of both the experimental and numerical models is shown in Table 1 for loading up to 4.6% drift. The benefits of FRP wrapping and of post-tensioning and uplift in the column are clearly shown in the large reductions in residual displacement. The remaining area of focus, the amount of hysteretic energy dissipation in the column systems, is summarised in Table 2. The base case is shown in the left-hand column, and as such it can be seen that there are reductions in the amount of hysteretic energy dissipation in all cases. Most notable is the significant reduction in energy dissipation of the PTFRP model as compared to both of the monolithic columns.

Table 1: *Residual Displacements/Drifts*

	Experimental	Numerical	
Monolithic	30.48/1.85	34.23/2.07	mm/%
PTFRP	4.09/0.25	0.08/0.01	mm/%
Mono FRP	-	16.1/0.98	mm/%

Table 2: *Differences in Numerical Hysteretic Energy Dissipation*

Base case	Comparison		
Monolithic	Mono FRP	-5.14	%
Mono FRP	PTFRP	-72.04	%
Monolithic	PTFRP	-73.81	%

4. CONCLUSIONS

Two 3D FE models have been developed and subjected to a cyclic loading pattern. The results were analysed and validated according to existing experimental data. The first model represents a conventionally built reinforced concrete monolithic bridge column, which forms a plastic hinge under cyclic loading, resulting in permanent damage and residual displacement. The second model utilises the more efficient precast concrete construction with FRP as a confinement material, and is allowed to uplift and develop a rocking motion under cyclic loading, which allows the structure to remain elastic and limits damage and residual displacements.

Preliminary modelling and a review of FE concrete models led to the use of two different concrete constitutive models for use in the FE models, dependent on the behaviour and amount of damage expected in the concrete. The built-in concrete element in ANSYS, SOLID65, was utilised in the monolithic model, with some modifications and the addition of a secondary kinematic hardening material model. In the PTFRP model, a solid structural element is defined, with a gradual softening behaviour imitated through the use of material changes with reduced strength properties according to the strain in each individual element. The results of the numerical models showed good agreement beside the existing experimental data, with the material models utilised resulting in an adequate representation of the concrete degradation under cyclic loading.

After validation of these models, a third model was developed consisting of a conventional monolithic column wrapped in FRP. The results of this simulation allow direct comparisons to be made between the three models such that conclusions may be drawn as to the benefits of FRP wrapping and rocking behaviour. Clear advantages can be identified in the potential of FRP to increase strength and reduce stiffness degradation and residual displacements as compared to conventionally reinforced columns. The post-tensioned rocking column further improved behaviour under sustained loading and reduced the permanent damage in the system with residual deformations of near zero. The rocking system was also able to withstand loading at large drifts whilst sustaining minimal damage. A disadvantage may be identified, however, in the form of reduced hysteretic energy dissipation in the rocking column, which would have to be improved before the system could be implemented.

With the advantages of rocking behaviour established, it will be possible to expand the existing validated 3D FE models to consider several different forms of increased energy dissipation in a more efficient manner than experimental studies. It will also be possible to extend the FE model to explore larger bridge span behaviour using rocking as an earthquake mitigation technique.

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