COMPDYN 2015
5th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Crete Island, Greece, 25–27 May 2015

SEISMIC ASSESSMENT OF THE FULL SCALE FOUR STOREY E-DEFENSE ROCKING SHEAR WALL SHAKE TABLE TEST

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Keywords: seismic assessment, reinforced concrete, prestressed rocking shear wall, nonlinear time history, nonlinear finite element.

Abstract. *An assessment of a four storey prestressed reinforced concrete structure is compared to existing results for a full scale shake table test. The E-Defense structure, which was tested in 2010 and was subjected to the Kobe earthquake record, is a complex concrete structure with prestressed rocking shear walls in one direction and a moment frame in the other. This work focusses on the seismic performance assessment of the rocking shear wall system subjected to the 1990 Kobe earthquake record at three intensity levels (25%, 50% and 100%), as was done in the shake table test. The purpose of this study is to assess if, rather than using heuristic approaches or experimental data to calibrate the nonlinear elements of a lumped plasticity model, a more advanced static nonlinear finite element procedure that can accurately capture the complexity of these elements could be used to calibrate the hysteresis of the plastic elements. In this study, the hysteretic response of the critical members, namely the rocking shear wall and the columns, were obtained through modelling in VecTor2, a two dimensional nonlinear finite element tool for reinforced and prestressed concrete structures. These hysteresis were then used to calibrate a nonlinear spring at the base of the shear wall in Ruaumoko and a full nonlinear time history analysis was conducted. The peak base shear and drift ratio at the peak were predicted well. Furthermore, the individual member responses, modeled in VecTor2, accurately captured the crack patterns and failure modes observed in the experiment. To mimic situations practitioners would encounter, the results were not adjusted after the analysis was conducted, the predictions were conducted in a blind prediction process.*

1 INTRODUCTION AND MODELLING APPROACH

Current techniques and modelling tools to assess the seismic performance of reinforced and prestressed concrete structures are vast. When engineers are faced with assessing the seismic performance of a large structure, decisions need to be made as to how complex and detailed the model will be. The analysis techniques under consideration may include simplified single degree of freedom models, linear modal analysis, linear fiber models, or more advanced lumped plasticity models, nonlinear fiber models, or full nonlinear finite element approaches. Contributing factors in deciding which approach to use involve with the tradeoff between the time required to implement the model, the complexity of the failure modes and the desired performance parameters to be assessed.

In 2010 Nagae *et. al.* [1] conducted a shake table test of a full scale four storey structure. The structure had a prestressed concrete rocking shear wall in one direction and a moment frame in the other. This paper outlines a blended approach to model the lateral load resisting system in the shear wall direction. The goal of the modelling was to conduct a seismic performance assessment that could be compared to experimental results. It was of particular interest to accurately estimate the peak base shear, the peak drifts, and the residual drifts and then to compare these results with the experimental values reported in the literature. To mimic conditions faced by practitioners, the results were not reviewed until after the full assessment was complete. Therefore, no aspects of the model were modified after the initial assessment was conducted.

One modelling approach to achieve this level of assessment for rocking shear wall systems, though not common in practice, is to use solid finite elements. Developing a full nonlinear finite element model of the entire structure is time consuming and computationally expensive [2]. The computational time for such a model may be on the order of hours or days. A common alternative is to use fiber models with zero tension capacity elements at the base of the shear wall [3, 4] so that the rocking mechanism can be captured. Similarly, lumped plasticity models can be implemented to establish predictions [2, 5]. Although fiber models and lumped plasticity models are significantly less time consuming to develop and run, establishing reliable hysteretic rules for the nonlinear elements becomes a critical part of the analysis. In situations where experimental results are not available or there is uncertainty surrounding the heuristic approaches used to estimate the hysteresis of the nonlinear elements, an issue commonly faced by practitioners, other procedures are needed to establish those parameters.

It was the intent of the authors to investigate the quality of an analysis that could be conducted with a lumped plasticity model but using a more advanced approach to estimate the hysteretic behaviour of the nonlinear elements. This blended approach has the advantage of being much less computationally expensive, the run time being several minutes rather than hours or days, and it provides a rational basis to develop the hysteretic rules. Specifically, the hysteretic behaviour of the rocking shear wall and the column elements were individually modelled using VecTor2 [6], a two dimensional nonlinear finite element model based on the Modified Compression Field Theory and the Disturbed Stress Field Theory [7, 8 and 9]. This blended modelling approach has the advantage that the VecTor2 modelling can accurately capture the full reverse cyclic load deformation responses of the elements, rocking mechanism, crack widths, prestressing, element stresses, crushing failure modes, shear failure modes and yielding failure modes. These elemental hysteresis were then used in a lumped plasticity model programmed in Ruaumoko [10] to conduct a nonlinear time history analysis. The global structure was subjected to the 1990 Kobe earthquake ground motion at 25%, 50% and 100% intensities, cumulatively, as was done in the 2010 E-Defense shake table tests [1]. Since the structure was subjected to progressively larger ground motions the cumulative damage needed to be accounted for in the modelling. As was previously mentioned the results were assessed in a blind manner to simulate conditions typical in industry.

2 STRUCTURE DETAILS

In the test conducted by Nagae *et. al.* [1] two structures were tested, a prestressed structure and a conventional reinforced concrete structure. Each test specimen had a shear wall in one direction and a moment frame in the other. This study is focused on the prestressed structure in the shear wall direction. Some details of particular interest include the detailing at the shear wall and corresponding prestressing. For instance, although the wall is a rocking system, some partially bonded mild steel bars were included at the base of the wall that were intended to yield and provide hysteretic energy dissipation. The prestressing tendons, which promote the self-centering properties of the wall, were unbonded.

The following details in Table 1, Figure 1 and Figure 2 were used to develop the VecTor2 and Ruaumoko models, further details can be found in the publication by Nagae *et. al.* [1].

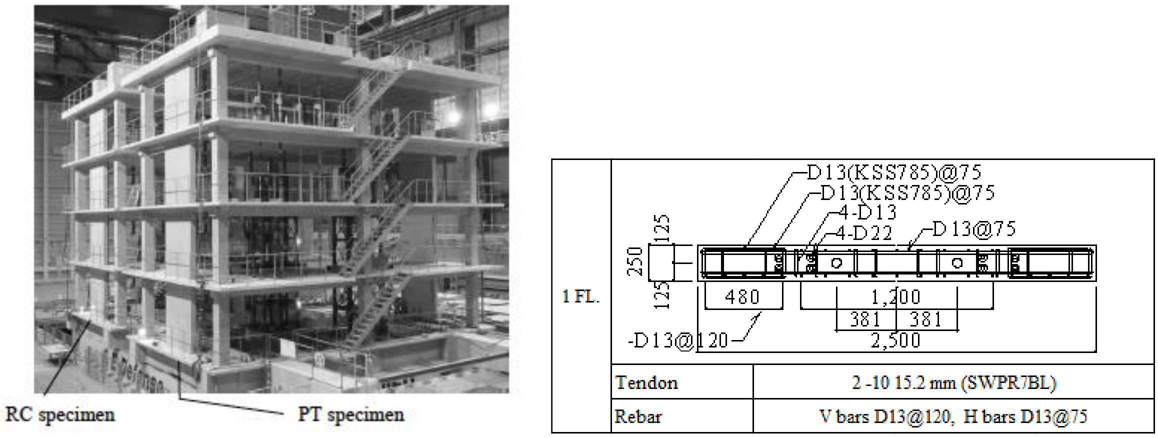

Figure 1: Photograph of experimental setup (left) and cross section of shear wall at the first floor (right), reproduced from [1]

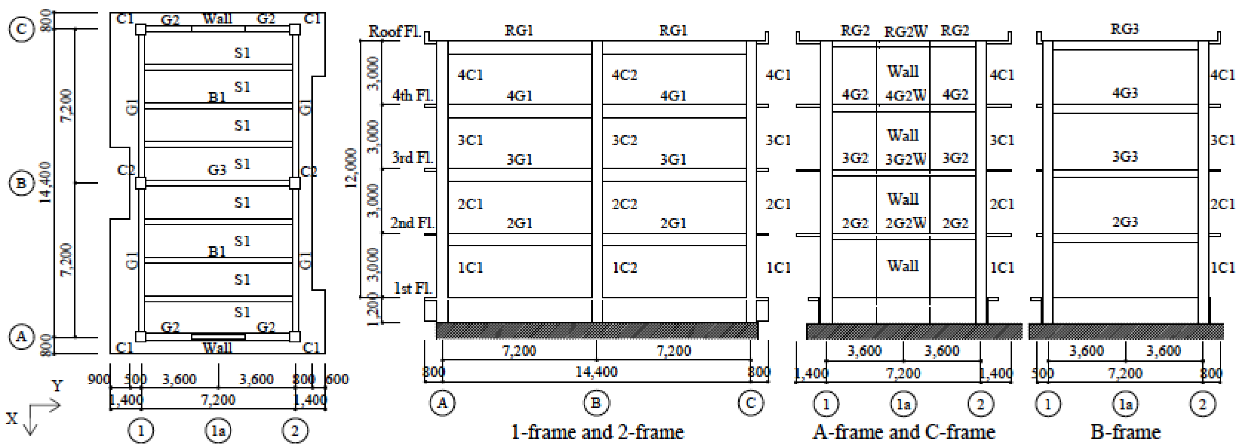

Figure 2: Elevation and plan of experimental setup, reproduced from [1].

	A_s (mm^2)	f_y (MPa)	f_u (MPa)
D22 Mild Steel	387	385	563
21 mm diam. PT Bar (column)	346	1198	1281
15 mm diam. PT Wire (wall)	140	1777	1969

Table 1: Steel material properties, reproduced from [1].

As was reported in the literature a concrete strength of 60 MPa was used.

3 VECTOR2 MODELLING

VecTor2 is a two dimensional nonlinear finite element tool capable of capturing the nonlinear behaviour of the cracked prestressed concrete rocking shear wall and the prestressed columns. Figure 3 shows the models used to develop the hysteretic responses.

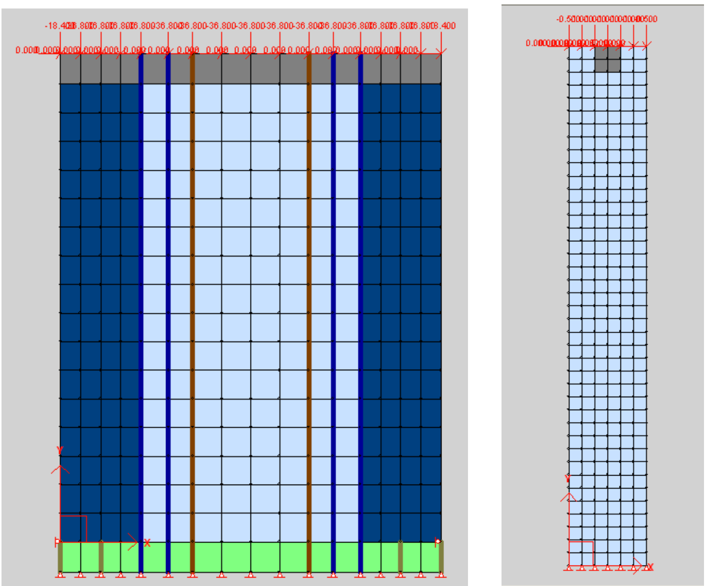

Figure 3: Finite element models for first storey rocking shear wall (left) and column (right).

In Figure 3, the dark blue regions in the model represent the interior columns of the shear wall with higher amounts of reinforcement (see Figure 1). The lighter blue regions represent the web region of the shear wall. The dark blue lines represent the partially bonded mild steel bars used by the designers as a means to increase the hysteretic damping. The brown lines are the unbounded prestressing strands, modelled as truss elements with the appropriate prestressing. The prestressing was implemented with a strain offset in the steel. Except the partially unbonded bars and the prestressing strands, the steel is modelled as smeared reinforcement. In this 3000 mm tall model of the first storey, the tendons are bonded to the top of the storey when in reality the tendons would be anchored to the top of the wall. Therefore, to ensure a uniform stress is introduced by the prestressing at the first floor, a row of stiff elements is used, they are shown in grey. The rocking is captured using a row of zero strength elements (green) and compression only truss elements (light brown). The stiffness of the compression only elements were set to approximate the stiffness of the foundation that was built using high strength concrete. The base of the structure was restrained from translating in the horizontal direction. The gravity load was imposed at the top of the wall. A simple pushover analysis was used to qualitatively verify the modelling assumptions, including mesh refinement. It was determined that a mesh made from approximately 200 mm x 200 mm elements yielded similar results to finer mesh sizes and therefore would be less computationally demanding when conducting the full static reverse cyclic analysis.

The gravity load was applied in the VecTor2 model and the model was loaded in a reverse cyclic manner with displacement control of the top node. Figure 5 in the following section shows the hysteretic response of the shear wall. This response was used to calibrate the rotational spring used in the Ruaumoko modelling. As can be seen the self-centering property of the shear wall is accurately captured.

Figure 4, obtained using the program Augustus [11], shows the magnified deformed shape and crack pattern as well as a map of the principal compressive stresses and the principal compressive stress directions (white lines) at the maximum predicted drift. The maximum crack widths are approximately 1 to 2 mm. It is also interesting to note that there are clear diagonal shear cracks in the wall. This is significant because using simpler modelling techniques would omit this significant phenomena. The blue elements in the figure showing the compressive stresses in the concrete indicate that crushing is occurring at the toe of the shear wall. This failure mode is typical for these systems and, in this case, did govern the failure. The dark blue elements have a compressive stress in the concrete of nearly 60 MPa and the light green elements have roughly 0 MPa.

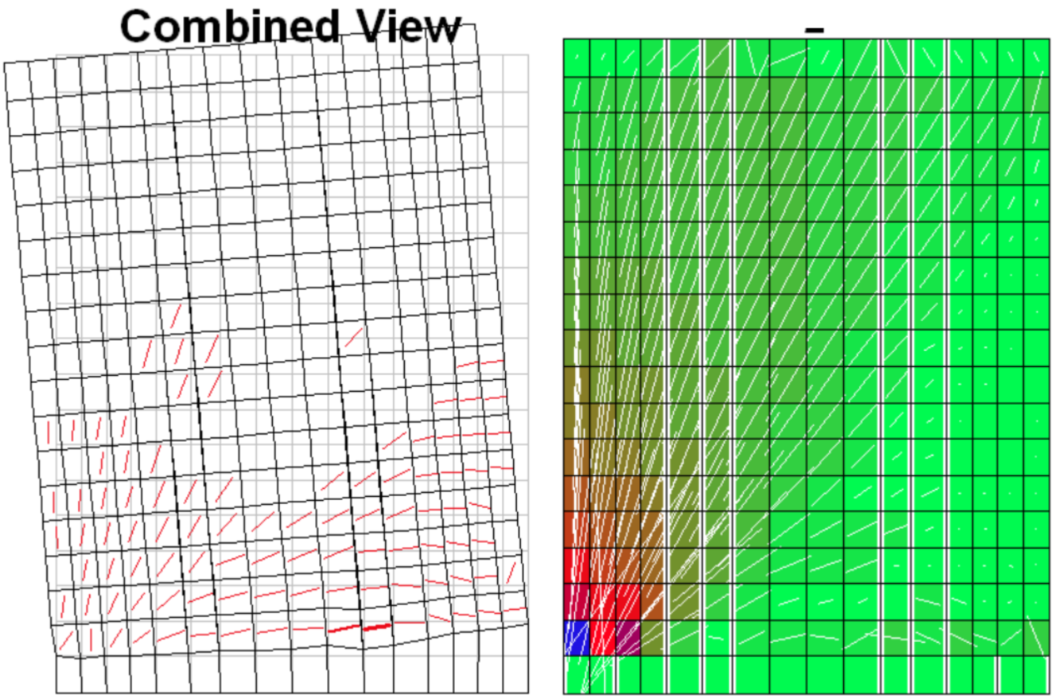

Figure 4: VecTor2 crack pattern and deformed shape (left) and principal compressive stress and principal compressive stress direction (right).

The columns were also modelled to account for prestressing; in this model all the reinforcement, including prestressing bars, were modelled as smeared, as can be seen in Figure 3. As with the shear wall, the prestressing was imposed with a strain offset in the steel, however, for the columns, the steel was bonded to the concrete since, in the experiment, the ducts were grouted. The rotation at the base was fixed and the top node was cycled in displacement control. The elements surrounding the node used for displacement control were strengthened to ensure a local failure did not govern the response (shown in grey). The gravity load appropriate to that floor was applied in the VecTor2 model. Figure 5 shows a typical hysteretic response of the column elements. Unlike the wall the hysteresis loops, the columns absorb less energy and although at large displacements the columns exhibit significant nonlinearity, at small displacements the response remains fairly linear.

4 NONLINEAR TIME HISTORY ANALYSIS AND THE RUAUMOKO MODEL

In 2010 the E-Defense structure that was tested by Nagae *et. al.* [1], the structure was consecutively subjected to three horizontal instances of the 1990 Kobe earthquake record with progressively larger intensities (25% PGA, 50% PGA and 100% PGA). The results of this experimental work is documented elsewhere [1, 12]. To model the global structural behaviour a two dimensional lumped plasticity model was developed in Ruaumoko. The rocking at the base of the shear wall was modeled with a rotational spring and the columns with plastic hinges that could develop at either end of the elements. The calibration of the nonlinear hysteretic rules for the rocking at the base of the shear wall and the columns were fit to the VecTor2 results. It was determined that since an estimate of the residual drift was an important part of the seismic performance assessment, a flag-shape hysteretic model best fit the results as it would ensure the 'snap through the origin' behaviour of the rocking system, see Figure 5.

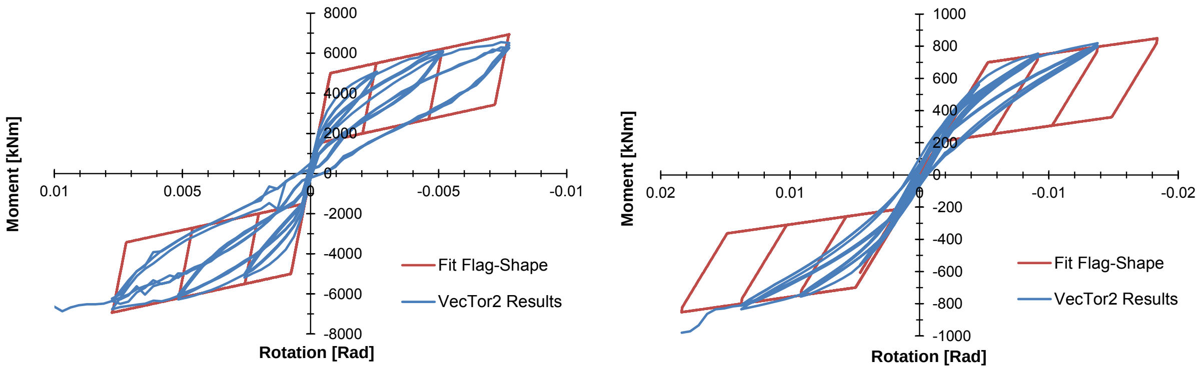

Figure 5: Shear wall (left) and column (right) hysteresis results from VecTor2 and element calibration.

For the shear wall, Taylor [13] beam-column elements that account for shear deformations were used. These elements use a Newton-Raphson iteration scheme to continually update the neutral axis based on the equilibrium requirements at every time step in the model. It was assumed the shear wall remained elastic above the base. Giberson beam-column elements [10] were used to model the columns. Similarly, the floor slabs were modeled as linear elastic and it was assumed that the connections remained rigid. The floor slabs were modeled with a thickness of 130 mm and with a width of half of the out-of-plane span. The elements were connected from the corner columns to the middle of the shear wall at each floor. Within the width of the shear wall (2500 mm) the elements are forced to rigidly deform with the wall, thereby capturing the correct shear span of the floor slabs. The damping was modeled by imposing a 5% damping ratio on all modes [14].

To account for possible P-Delta effects a set of dummy columns were used and slaved to the structural system. These column elements are pinned at both ends at every storey and the full gravity load of the structure is applied. Since these elements have no lateral resistance, as the structure deforms the main lateral load resisting system is needed to resist the P-Delta effects that develop. Figure 6 (left) shows a drawing of the Ruaumoko model for the structure. The hatched areas are the rigid links used to account for the effect of shear wall width. The red elements are the floor slabs, the peripheral blue elements are the columns and the blue elements in the center, are the shear wall elements. A depiction of the system modelling is shown in Figure 6 (right). Here, the column to the right of the graphic represents the dummy columns used to capture the P-Delta effects.

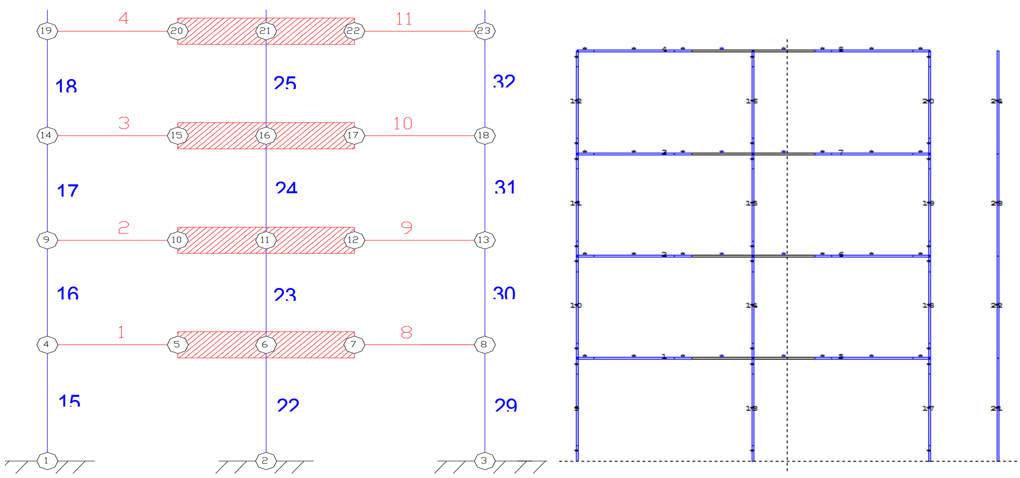

Figure 6: Ruaumoko model.

5 RESULTS OF NONLINEAR TIME HISTORY

The results of the nonlinear time history analysis from the blind Ruaumoko seismic assessment were compared to experimental results from the literature. Table 2 summarizes some of the key parameters.

	T_n (s)	V_{max} positive (kN)	V_{max} negative (kN)	Δ_{max} roof (mm)	$\Delta_{residual}$ roof (mm)
Experiment	0.29	3000	2950	204	80
Prediction	0.31	3274	3823	125	11

Table 2: Summary of analysis results.

As shown in Table 2, the prediction of the period was excellent. The prediction of the maximum positive peak was also good with a test to predicted ratio for the base shear of 0.92. The base shear for the maximum negative peak is somewhat over predicted. Although the overall shape of the column and shear wall hysteresis are captured with a flag-shape, not enough energy is dissipated at lower loads. Since the structure underwent two lower intensity earthquakes prior to the 100% record, additional damage would have occurred and lowered the stiffness of the structure for the final record. Although these records were applied in the model, the flag-shape hysteresis provides no dissipation and would remain completely undamaged before the yield point. Furthermore, it is possible that the hysteretic damping used in the Ruaumoko model is underestimated. Together, these differences resulted in an over prediction of the maximum base shear on the negative peak. The prediction of the maximum roof drift and residual drifts are also underestimated. This is likely caused by the fact the flag-shape hysteresis that always passes through the origin. Once the partially bonded, mild steel bars yield and shear cracks develop the hysteresis does not pass directly through the origin, as is predicted in the VecTor2 models but unlike the assumed hysteresis.

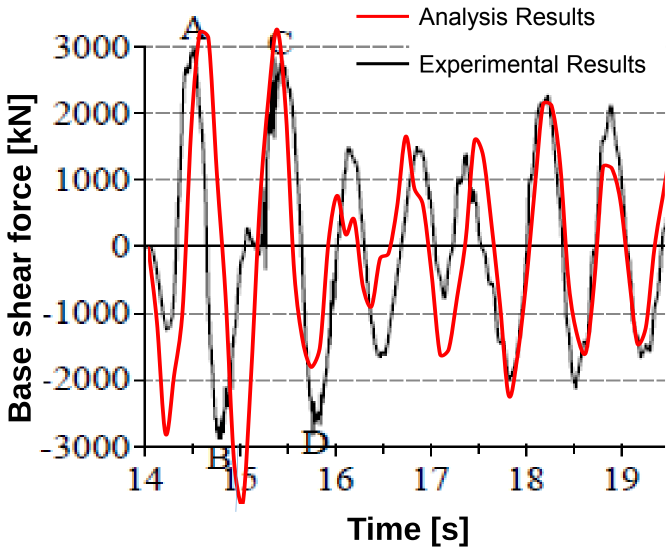

Figure 7: Comparison of the observed and predicted behaviour near the peak response, reproduced in part from [1].

The overall trends for the dynamic response near the peak, shown in Figure 7 and Figure 8 are good considering these results were established in a blind prediction manner.

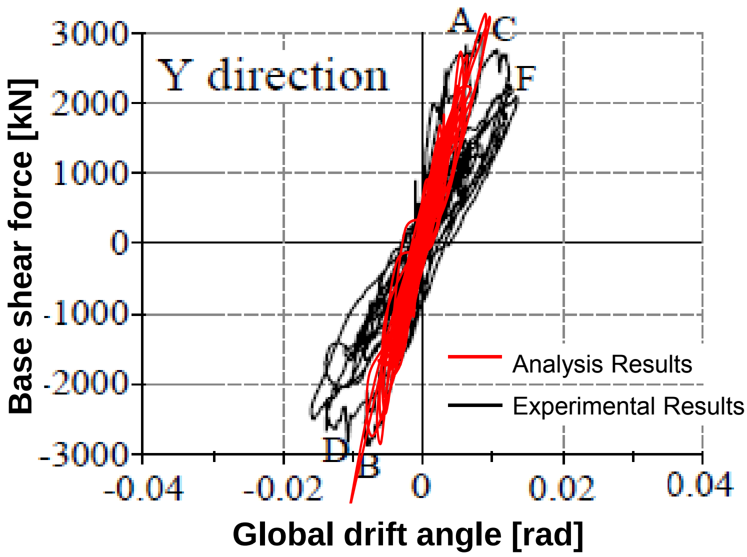

Figure 8: Global hysteretic behaviour, reproduced in part from [1].

Figure 8 shows the structural response in comparison with the experimental results. This plot corroborates the conclusions that the model did not undergo sufficient damage, for the reasons discussed previously. Specifically, unless the yield point of the flag hysteresis is reached no energy dissipation or damage occurs, which results in the discrepancy between the predicted and observed behaviour. This suggests that although the overall appearance of the flag-shape hysteresis used to model the nonlinear elements was reasonable, it is important that: the amount of energy dissipation is closely matched for all load levels, the gradual stiffness degradation is captured and the damage caused by unloading at all load levels can be captured [15]. This is particularly important in this analysis since prior to the 100% intensity Kobe record was applied two other lower intensity records, the 25% Kobe and 50% Kobe were applied.

6 CONCLUSIONS

In modelling the seismic performance of a complex prestressed concrete structure there are benefits to using advanced nonlinear finite element techniques to calibrate the plastic elements in the global model. Using this approach, a relatively simple and computationally inexpensive lumped plasticity model can be used to conduct the nonlinear time history analysis. Although a nonlinear finite element model is required for the element calibration, it avoids the necessity of using generic heuristic methodologies or experimental results based on element categories. In using the VecTor2 model, the peak capacity, ductility and an estimate of the hysteretic dissipation can be rationally estimated. It also ensures that complex failure modes, such as shear failures, crushing failures and the rocking mechanism are accurately captured.

The estimate of the first positive peak shear force was captured with good accuracy with a test to predicted ratio for the base shear of 0.92. The peak base shear at the first negative peak was somewhat over estimated. This is likely a result of the flag-shape hysteresis used. In particular, in the linear portion for the hysteresis before yielding, when the structure unloads, no energy is dissipated. Since the actual hysteresis determined from VecTor2 is highly nonlinear throughout the loading and unloaded phases it is difficult to capture using piecewise linear relationships. For similar reasons, the peak roof displacement and residual displacements are somewhat under predicted by this model. It should be further investigated if a rational unloading rule can be programmed into the Ruaumoko model to better capture: the amount of energy dissipation at all load levels, the gradual stiffness degradation and the damage caused by unloading at all load levels. It appears that these aspects are significant. It should be noted that to objectively evaluate this modelling approach a 'blind prediction' methodology was employed and therefore, the results were reasonable. That is, the VecTor2 model, Ruaumoko model and hysteretic behaviours were not adjusted based on experimental data whatsoever before comparing the results to those in the literature. This methodology should be employed in assessing new modelling techniques as it provides a realistic basis of how the approach would perform in a practical environment.

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