# Nonlinear seismic assessment of lightly reinforced concrete intake towers

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SYNOPSIS. Published guidance on the seismic analysis of reinforced concrete intake/outlet towers is limited, especially for their nonlinear response, due to limited knowledge on the nonlinear characteristics of existing and new towers. Proving the integrity of existing towers is an international problem for dam owners, and an industrial need exists for a rational, cost-effective and validated method for their assessment.
This paper describes a series of tests aimed at investigating the seismic performance of typically reinforced, non-seismically designed towers. Monotonic and cyclic push-over tests were performed on 1/6$^{th}$ scaled models. The results from the physical tests were used to validate a 3D nonlinear finite element model of the towers, using embedded steel reinforcement and a smeared crack model to simulate crack properties of the concrete material. The dynamic performance of the structures was investigated by developing a simplified single degree of freedom model and performing a number of simulations to obtain fragility curves of the system. This simplified model was capable of simulating the degrading, hysteretic properties of the towers and was used to perform nonlinear time history analyses using a range of parameters. A probabilistic approach was selected as the basis of the performance evaluation process using fragility analyses as a tool for modelling the uncertainty associated with the parameter selection. Based on the experimental and analytical results, a three-staged assessment procedure for the seismic performance assessment of the towers was proposed.

## INTRODUCTION

Intake/outlet facilities (Figure 1) form part of the vital infrastructure of a dam as they regulate the outflow of water from the impounded reservoir. In the event of an earthquake occurring, it is therefore essential that any damage to the intake tower does not induce the catastrophic failure of the dam, and consequent release of water. Continued operation of the facility may also be required to allow controlled release of water to permit essential repair work to be carried out if damage occurred to the main barrage itself. The seismic risk to dams in the UK has been studied extensively, resulting in the publication of design guides (Charles et al., 1991; Institution of Civil Engineers, 1998). However, limited guidance is available for specifically assessing the seismic vulnerability of intake towers (ICOLD, 2002; USACE, 2003). Existing seismic design codes, such as Eurocode 8 (BSI, 2004) provide limited guidance for their application to intake towers.

The purpose of this research was to determine the characteristic nonlinear behaviour of typical UK lightly reinforced concrete intake towers under seismic loading. This was done through a series of experimental and theoretical investigations into the nonlinear behaviour of scaled intake tower models subject to monotonic and cyclic pushover loads, leading to the development of a simplified probabilistic tool as part of a rational method for the evaluation of their seismic performance. By establishing appropriate performance requirements for given limit states, the seismic response of the towers was evaluated in a probabilistic context.

Figure 1: View of Errochty tower and access bridge

## EXPERIMENTAL MODELLING OF INTAKE TOWERS

A desk study of the typical characteristics of existing reinforced concrete intake towers in reservoirs in the United Kingdom was undertaken. By averaging the typical values for geometry, reinforcement steel and material properties, a prototype tower configuration was obtained. For the purpose of this project, a rectangular hollow free-standing tower was selected. The area of reinforcement steel to area of concrete ratio (ρ) was chosen as 0.25% for both vertical (longitudinal) and horizontal (secondary) reinforcement, representing typical UK values. The control house, access bridge and other appendages, as well as the water-structure and soil-structure effects, were not considered as the primary purpose of the experimental programme was the understanding of the structural response to earthquake type loading only. Table 1 summarises the prototype and scaled model geometrical properties. The choice of the 1:6 scale was based on a compromise between practical size for testing, cost and the ability of adequately replicating the failure behaviour of the intake tower.

Table 1: Prototype and scaled model geometry

| Tower geometry | Height (m) | Width (m) | Wall thickness (mm) |
|---|---|---|---|
| Prototype | 18 | 6 | 600 |
| Scaled model | 3 | 1 | 100 |

Two intake tower specimens (NSD-R-1 and NSD-R-2) were constructed in the Earthquake and Large Structures Laboratory (EQUALS), part of the Bristol Laboratories for Advanced Dynamics Engineering (BLADE) testing facilities at the University of Bristol. The specimens were constructed as ultimate strength, or replica, models (Harris and Sabnis, 1999) using model concrete and model reinforcement materials which satisfied the similitude conditions for the prototype materials.  Full details are given in Sabatino (2007).

Model material characteristics

The correct modelling of the materials ensured the performance of the model under quasi-static loading to adequately replicate the behaviour of the prototype. For successfully modelling the correct failure mode of the structure, and in particular distinguishing between brittle and ductile failure, it was necessary to develop model materials which would satisfy the similitude requirements of cracking, bond and strength – the parameters which govern the nonlinear response at a local level. Therefore, stress and strain characteristics of the materials were not scaled down.

The model concrete was developed using typical constituent materials for ordinary concrete: cement, sand, grit, chippings and water. However, the a reduced aggregate size was used. Steel reinforcement was modelled using 4mm cold-rolled threaded bars, or studding, which were heat treated to obtain suitable constitutive stress-strain characteristics.

Monotonic and cyclic push-over tests of scaled models

Two quasi-static push-over tests on $1/6^{th}$ scale intake tower models were carried out in order to determine their load-displacement properties, in particular their capacity and cyclic degradation characteristics (Figure 2). The test specimens were mounted onto a purpose built reaction frame and subjected to either monotonic (specimen NSD-R-1) or cyclic (specimen NSD-R-2) push-over loads. Four servo-controlled hydraulic actuators were used to impose horizontal and vertical loads. The lateral loading was representative of earthquake loading, whereas the vertical loading was used to simulate approximately the added mass for gravity similitude. The towers were instrumented with a range of strain gauges and displacement transducers designed to record data describing the response of the structure to the applied loads. Approximately 80 channels of data were recorded.

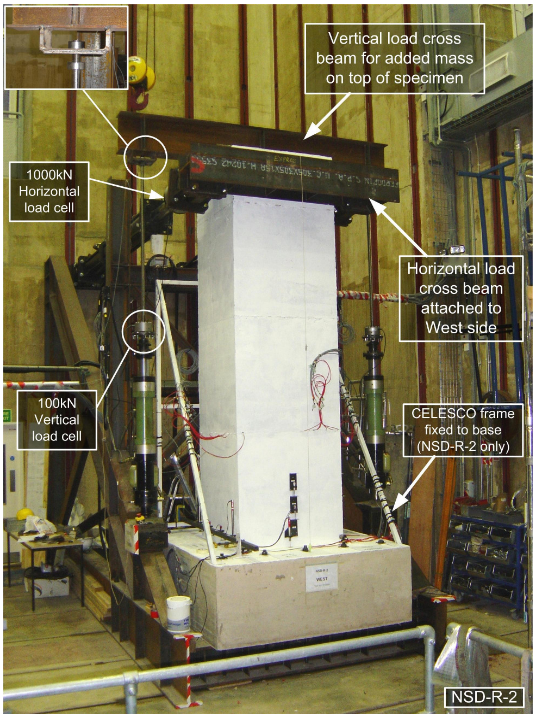


Figure 2: Tower specimen setup

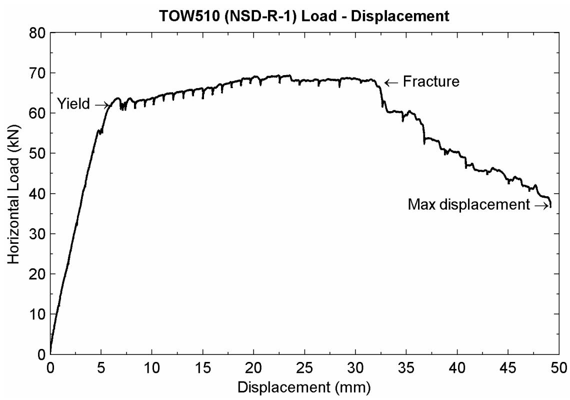


Figure 3: Load-displacement plot for monotonic test (NSD-R-1)

*Test results*

Monotonic test results indicated that the response of specimen NSD-R-1 was characterised by a tension failure of the under-reinforced section, with a single large crack forming at the base of the tower. With reference to the load-displacement plot (Figure 3), three distinct phases were observed:

1. The response of the specimen was stiff until first cracking was observed; first yielding of the reinforcement followed.
2. As further displacement was imposed, some hardening was observed until the peak load was reached.
3. Once fracture was initiated, the reaction force to the applied displacement dropped rapidly until the test was terminated.

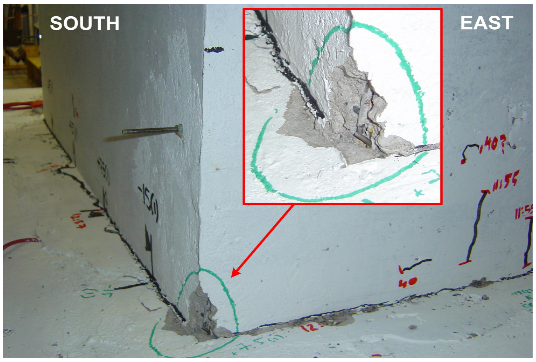


Figure 4: View of South-East corner crushing (NSD-R-2)

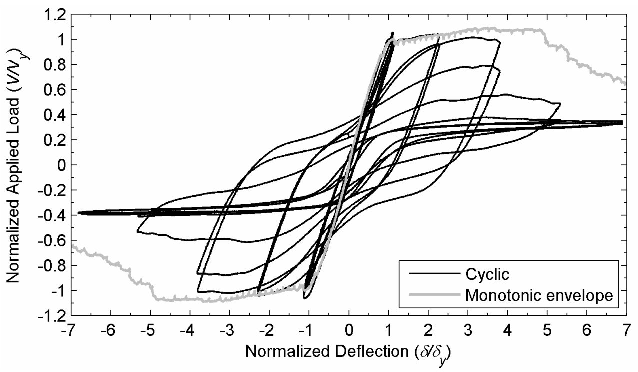


Figure 5: Normalized load-displacement plot.

The cyclic response of specimen NSD-R-2 confirmed the rigid body behaviour, with localised damage at the base including spalling of the concrete cover due to bar buckling (Figure 4). The following additional conclusions can be obtained from the cyclic test results:

1. Considerable strength and stiffness degradation was observed with increasing displacement amplitudes, making the monotonic envelope (Figure 5) a non-conservative estimate of the tower capacity.
2. Extreme pinching was likely to be caused by bond deterioration between the steel reinforcement and the concrete.

3. Yielding of the reinforcement was localised along the critical section, with small (elastic) strains being measured at other locations along the height of the tower.

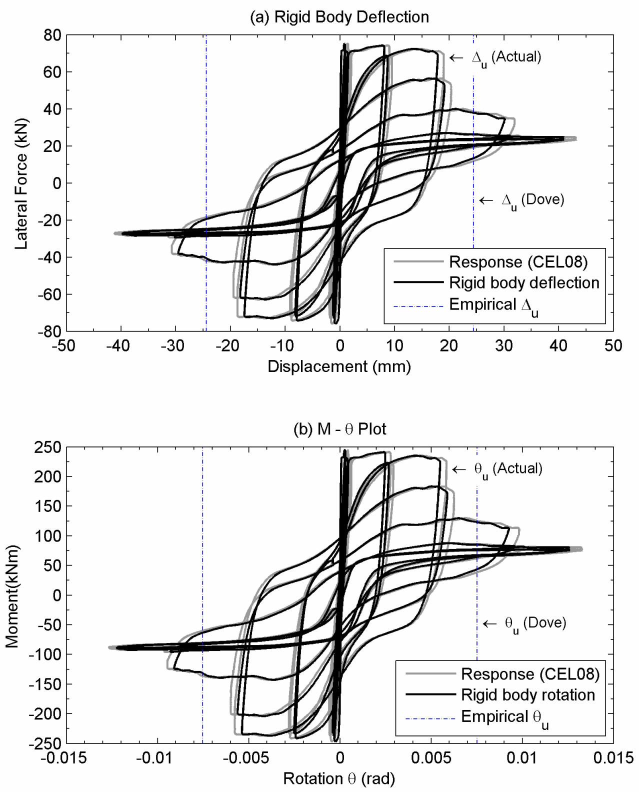


Figure 6: Experimental and empirical (a) ultimate deflection and (b) ultimate rotation values

Table 2: Comparison between experimental and predicted ultimate deflection and rotation values

| | Actual (experimental) capacity | Theoretical capacity | Error |
|---|---|---|---|
| Ultimate deflection $\boldsymbol{\Delta}_u$ | 18.9 mm | 24.4 mm | 29.1% |
| Ultimate rotation $\theta_u$ | 0.005832 rad | 0.007514 rad | 29% |
| Ductility ratio $\mu$ | 21 | 28 | 33% |

Figure 6 shows the ultimate deflection ($\Delta_u$) calculated using the empirical expression proposed by Dove and Matheu (2005). The ultimate rotation ($\theta_u$) was calculated, based on the rigid body assumption, by dividing $\Delta_u$ by the height. Both plots (a) and (b) show that the actual ultimate deflection and rotation of the specimen, obtained from the pushover tests, are less than the calculated empirical value. For this configuration, the predicted ultimate deflection (or rotation) is over-estimated using the empirical relation. This has considerable consequences on the ductility ratio of the tower (Table 2).

## NUMERICAL MODELLING OF INTAKE TOWER SPECIMENS

### Nonlinear finite element model

A Finite Element Analysis (FEA) of the intake tower specimens was undertaken to investigate the suitability of the method for modelling typical lightly reinforced concrete intake towers under seismic loading by comparing the numerical results with the benchmark experimental results. The modelling of the intake towers was performed using DIANA (release 9) finite element package, a general purpose commercial finite element code based on the displacement method (DIANA, 2006).

*3D model definition*

The finite element model geometry was based on the three-dimensional properties of the physical specimens, with the actual steel reinforcement layout modelled. The horizontal (monotonic and cyclic) load was applied in the form of explicitly specified load steps through the definition of a time curve. The vertical added mass was modelled as a point load applied at the top of the model. The concrete cracking was modelled numerically using the smeared crack approach to allow for a more versatile finite element model. The total strain fixed crack model (DIANA, 2006) was selected for its ability to formulate a single model with tensile and compressive constituent laws. The steel reinforcement was modelled as an elasto-plastic material with no ultimate strain defined.

*Monotonic and cyclic analysis results*

The duration of the monotonic analysis was 24 hours for 178 load steps, using a dedicated Windows server. The analysis results indicated a localised crack occurring at the base of the tower (Figure 7), with the rest of the model remaining within the elastic range. Figure 8 shows the load displacement results compared to the experimental results and the rigid body motion calculated empirically. The FEA results coincide closely with the rigid body motion, whereas the discrepancy with the experimental results was due to the error in the measurement of the lateral displacement of the tower, affected by the flexibility of the support frame.

The duration of the cyclic analysis was 24 days for 1279 load steps. The cyclic load history was defined to simulate the experimental cyclic loading.
Figure 9 (a) shows the horizontal load against applied displacement plot for the top of the model. By comparison to the experimental results (grey curve), plotted for the same amplitude displacements, the numerical results slightly under-estimate the capacity of the tower, probably due to some difference between the numerical concrete strength and the actual tower

concrete properties. However, the magnitude of the displacement and the unloading/reloading stiffness of the tower appear to coincide with the experimental results.

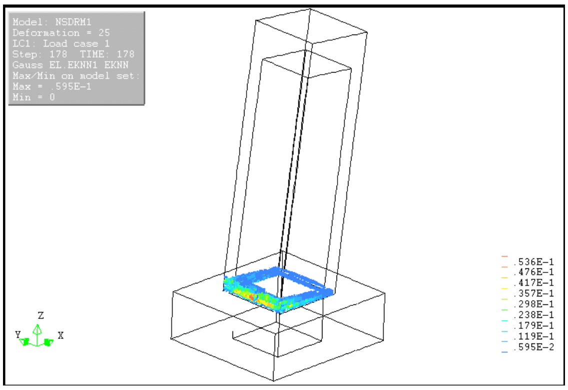

Figure 7: Monotonic crack distribution at time step 178 (f)

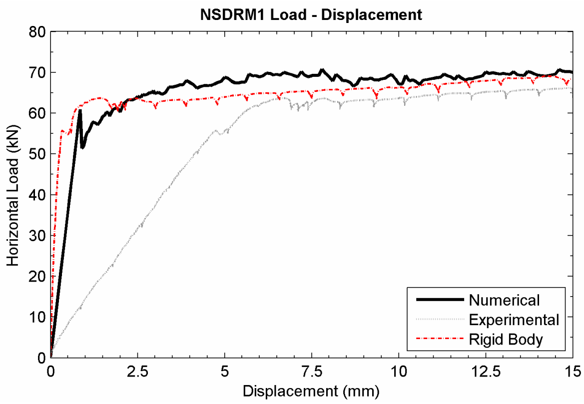


Figure 8: Monotonic load-displacement curve

Figure 9 (b) shows the plots, for both numerical and experimental results, of the normalized load, obtained by dividing the load by the yield value, and the normalized displacement, calculated by dividing the displacement by the yield value. The figure shows a good relationship between numerical and experimental results, suggesting that the FEA model is capable of adequately simulating the capacity of the structure, although strength degradation during the final cycle is more apparent for the experimental results.

Once fracture of the steel occurs in the physical model, this correlation is reduced as the numerical steel model does not allow for fracture of the steel to occur, emphasising the importance of defining the actual constitutive material characteristics in order to predict the full nonlinear response
Overall, the following conclusions may be drawn from the FEA results:

- The similitude between experimental and numerical results, in particular the crack distribution, indicate that the smeared crack approach is suitable for investigating the nonlinear response of lightly reinforced concrete towers.
- Shear was modelled explicitly using a shear retention factor. The analysis results were sensitive to the choice of the shear retention factor, even though the failure mode of the structure was tension. Careful consideration needs to be taken in selecting the crack model parameters to avoid a stiff response where it does not occur.

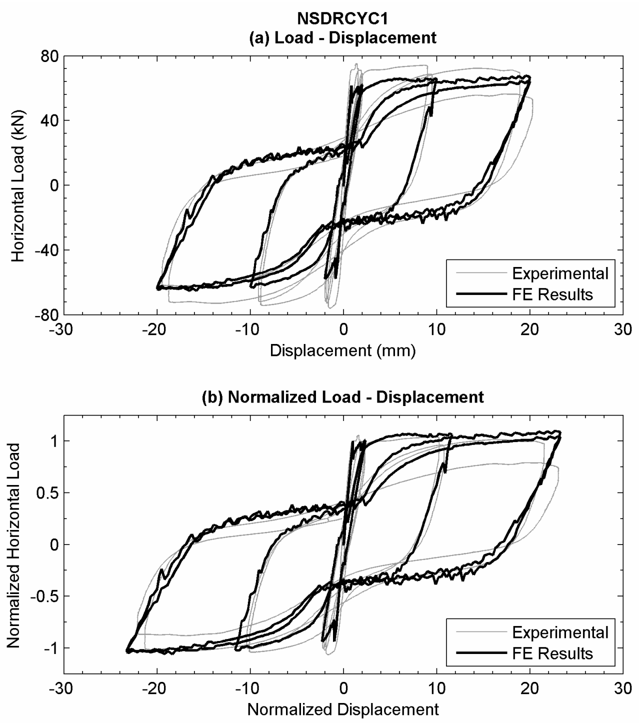


Figure 9: Cyclic load-displacement plots for numerical and experimental results: (a) actual; (b) normalized to yield values

Overall, the FE model showed a good simulation of the cyclic behaviour of the tower, but was impractical for use in probabilistic context due to the computational effort required. However, the results were used to validate the experimental results.

<u>Simplified model for dynamic time history analysis</u>

The response of the tower specimens described above suggests that the squat, lightly reinforced concrete towers may be approximated to a rigid block. The shaft section may therefore be modelled as a concentrated point mass, connected to a rotational spring modelling the crack opening at the critical section. The system lends itself to a single degree of freedom

(SDOF) idealisation, which is particularly advantageous for dynamic analysis of a nonlinear system, where the seismic action is represented as an acceleration time history. The hysteretic behaviour of a system can then be modelled by defining the characteristics of the SDOF spring to represent the nonlinear constitutive response of the structure. The SDOF idealisation (Figure 10) was used as the basis of a probabilistic approach to investigate the seismic vulnerability of various intake tower structures, thus allowing for the uncertainty associated with the selection of the parameters defining both structure and earthquake ground motion to be adequately represented.

*SDOF model definition*
An adapted Bouc-Wen model (Bouc, 1967; Wen, 1976) was chosen as a suitable mathematical representation of this relationship due to its versatility in defining the governing parameters. The model was adapted to simulate accurately the nonlinear response of the towers to earthquake-type loading, including structural degradation and pinching effects, and was calibrated against the experimental test results of the specimens. Details of the simplified model derivation are given in Sabatino (2007).

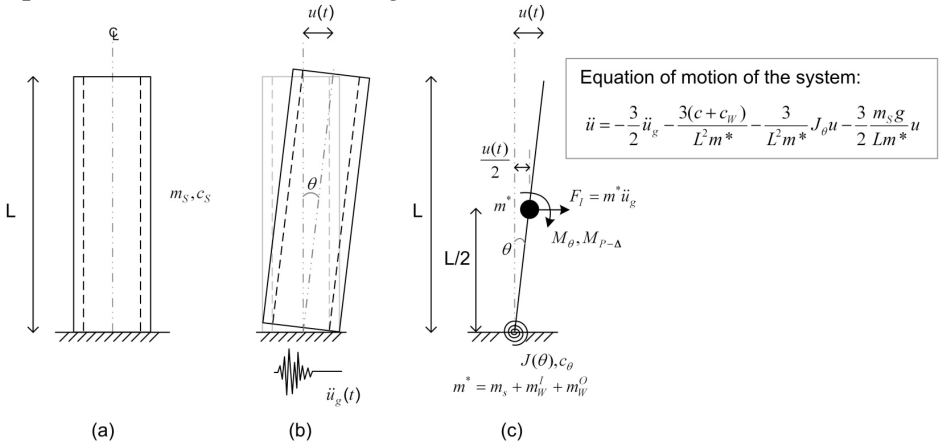


Figure 10: (a) Intake tower; (b) Rigid body response to earthquake; (c) SDOF idealisation

*Earthquake ground motions*
For the purpose of demonstrating the validity of the concept, synthetic earthquake ground motions were generated using the Kanai-Tajimi power spectrum (Tajimi, 1960) to represent typical UK, short duration earthquakes. The time histories generated for this study were deficient in energy content at very low frequencies. The results obtained should therefore not be viewed as meaningful in describing the real seismic performance of the intake towers.

*Fragility analysis*

The limit states considered in assessing the seismic performance of the towers were based on damage limitation (limit state 1 – LS1) and near collapse (limit state 2 – LS2) performance levels, as prescribed by Eurocode 8 Part 3 (BSI, 2005). By using the displacement ductility ($\mu$) as a measure of the performance level, values of $\mu$ were determined for first yield ($\mu_{LS1} = 1$) and ultimate displacement at fracture initiation of the reinforcement steel ($\mu_{LS2} = 20$).

Results from the dynamic analyses showed that the response of the towers was dominated by their inertial characteristics. The stiff structures dissipated little energy in their elastic range and only after cracking was initiated was the dominant response governed by plastic yielding, with the displacement of the SDOF system proportional to the force applied.

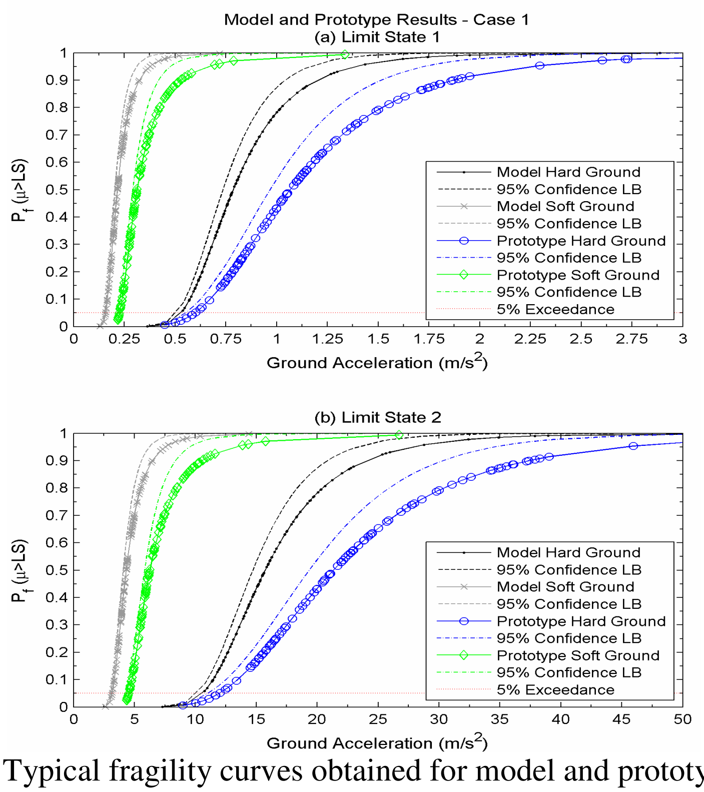


Figure 11: Typical fragility curves obtained for model and prototype towers (reservoir full conditions)

Indicative fragility curves describing the seismic performance of the structural systems considered were obtained (Figure 11). The main conclusions drawn from the fragility analyses are summarised below:

- The added mass contribution due to the reservoir water increases the seismic vulnerability of the system by approximately 15% for hard ground conditions.

- The response of the structures was sensitive to the shape and amplitude of the time histories used. The energy content of the earthquake, as well as the frequency content of the time history, greatly influenced the seismic performance of the structural models considered.
- The apparently large *PGA* response required to reach the collapse prevention limit state can be attributed to the geometrical characteristics of the towers, where large displacements are required before the centre of gravity of the structure is sufficiently displaced to induce overturning P-Δ effects. Although all the reinforcement steel may be fractured, the squat structure would respond in a rigid body rocking motion, requiring a large *PGA* to cause it to overturn.

Due to time constraints, the earthquakes generated for the purpose of this study were very crude. It was evident that a more precise evaluation of the seismic event, preferably through the generation of site specific time histories or at least by using more refined stochastic methods in simulating ground motions allowing for the appropriate ground characteristics to be modelled, would be required to obtain any significant performance assessment of a real structure.

These conclusions have been drawn from a simplified model which has been calibrated against quasi-static test results and a number of simplifying assumptions have been used. The validity of the dynamic response of the model requires further investigation, either through dynamic testing of tower models or, preferably, through comparison with time history analyses using more refined models over a range of parameters.

## PROPOSED STAGED ASSESSMENT PROCEDURE FOR SEISMIC PERFORMANCE EVALUATION

The proposed staged assessment procedure, described below, allows for a gradual increase in the complexity of the analysis where necessary.

***Stage 1:*** An initial performance assessment of the tower can be carried out using existing capacity spectrum methods. The capacity curve of the tower can be readily estimated using commercial packages, and the capacity spectrum method allows for an initial estimation of the whether a given seismic demand is likely to exceed the tower's capacity and therefore require a more rigorous nonlinear dynamic analysis.
***Stage 2:*** Using a simplified model, a probabilistic, second stage analysis would follow if necessary. A number of simulations can be carried out to obtain fragility curves for the structure for various loading conditions and performance requirements. The Engineer can then determine the vulnerability of the tower for a given seismic hazard level, and establish whether there is the need for a more detailed, and costly, FE analysis.

***Stage 3:*** A nonlinear FE analysis would only be required for those cases where, based on the fragility analysis results from Stage 2, the seismic demand exceeded the capacity of the tower. By selecting a few time histories, identified in the previous stage as having the most significant impact on the response of the tower, the dynamic time history analyses of the tower would provide a more accurate estimate of the structural performance.

## CONCLUSIONS

Monotonic and cyclic pushover tests of scaled lightly reinforced intake tower models indicated that their response was characterised by a localized tensile failure at the base of the tower. The rest of the tower behaved as a rigid block, with negligible flexural response. Significant strength and stiffness degradation were observed for the cyclic test, with the monotonic envelope providing a non-conservative estimate of the capacity of the structure. The response of the towers indicated that the empirical method proposed by Dove and Matheu (2005) to calculate the ultimate deflection capacity of intake towers over-estimated the actual capacity of the specimens tested. Further experimental studies would be needed to clarify this issue.

The test results were compared to a FE model of the specimens, capable of simulating the crack propagation using a smeared crack material model for the concrete. The computational effort required to run the analyses made this approach not viable for use in a dynamic time history analysis. A simplified SDOF idealisation was developed and used to perform Monte Carlo type analyses of the towers for different loading conditions and performance levels.

Based on the results obtained, the proposed simplified model was shown to be a credible tool for the dynamic analysis of lightly reinforced concrete intake tower structures. However, gross simplifications have been assumed in defining both the structure and the synthetic earthquakes used in the analyses. Although it provides a useful tool for the seismic vulnerability assessment of the towers, it should be used as a first approximation. The good fit to the quasi-static cyclic response of the actual towers does not guarantee that the dynamic simulation accurately matches the behaviour of a real tower. The uncertainty associated with the variability of the parameters defining the structure and earthquake, as well as the sensitivity of the response to the energy content and shape of the time history, implies that a risk-based approach should be adopted for assessing the performance of the intake towers.

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