

Free Vibration Characteristics of a Quarter-Scale Reinforced Concrete Frame: Experimental–Model Comparison

Dr. Mohammad Arastu

Structural Engineer
Jamia Millia Islamia University

Abstract

This paper examines the reported fundamental frequencies of a quarter-scale, four-storey reinforced concrete frame following successive shake-table runs. The experimental programme used a one-bay frame and a scaled 1940 El Centro excitation with nominal peak ground acceleration labels from 0.4 to 2.0 g. Post-run frequencies obtained from roof free-vibration records are compared with mathematical-model values supplied in the manuscript. The measured frequencies decrease from 8.10 to 7.44 Hz across the reported sequence, corresponding to an 8.15% decrease and a period increase from 0.1235 to 0.1344 s. The model frequencies are consistently higher than the measurements, with relative differences of 2.02–4.94%, a mean absolute error of 0.214 Hz, and a root-mean-square error of 0.234 Hz. Reported table peaks differ materially from the nominal input labels, so achieved and nominal excitation levels are distinguished. The comparison is descriptive: model formulation, updating procedures, and measurement uncertainty are insufficiently documented to establish independent model validation. The experimental observations overlap with an earlier published study and are identified as previously reported data. The revised interpretation quantifies frequency agreement without treating the observations as a new experimental campaign.

Keywords: reinforced concrete frame; free vibration; shake-table testing; fundamental frequency; experimental–model comparison; secondary analysis.

1. Introduction

Dynamic characteristics provide a compact description of structural response. For a reinforced concrete frame, measured frequencies depend on stiffness, mass distribution, boundary conditions, and the state of the specimen. Comparing experimental frequencies with model outputs can reveal discrepancies, but agreement in a single response quantity does not establish the accuracy of the complete structural model [1, 2].

Combined numerical and shake-table investigations have been used to assess structural models at different scales. Zhou et al. [3] compared software predictions with experiments for a complex connected building. More recently, Nakipoglu and Donduren [4] examined the effects of damage and strengthening on reinforced concrete frame dynamics. These studies support the use of experimental comparison while also showing why the specimen, loading sequence, and modelling assumptions must be documented.

The present manuscript considers the frame programme reported by Arastu and Moin [1]. Its focus is the comparison of five post-run measured frequencies with five mathematical-model values retained from the supplied manuscript. The contribution of this revision is a transparent, quantitative interpretation of those reported values; it does not introduce new tests or a newly executed structural analysis.

2. Specimen and experimental arrangement

The specimen was a quarter-scale, one-bay, four-storey reinforced concrete frame tested on a 2 m × 2 m shake table. Column spacing was 1,750 mm in both plan directions and storey height was 750 mm. The frame was cast in situ. The principal dimensions and reinforcement reported in the source manuscript are summarised in Table 1.

Table 1. Reported specimen properties.

Component or property	Description
Beams	75 mm wide × 100 mm deep; two 8 mm bars at both top and bottom
Columns	100 mm × 100 mm; four longitudinal 8 mm bars
Transverse reinforcement	2 mm wire; 25 mm spacing near joints and 50 mm elsewhere
Footings and base plates	300 × 300 × 100 mm footings; 300 × 300 × 8 mm plates
Anchorage	Four M12 bolts at each column base
Steel yield strengths	Longitudinal: 500 MPa; transverse: 250 MPa (reported)
Microconcrete strength	Average compressive strength: 25 MPa at 7 days

For beams, the closer stirrup spacing extended over a length of twice the beam depth from the column face. The bolted arrangement was intended to restrain the specimen at its base. Actual support flexibility, bolt preload, added floor masses, and the correspondence between experimental and numerical restraints are not quantified in the supplied record.

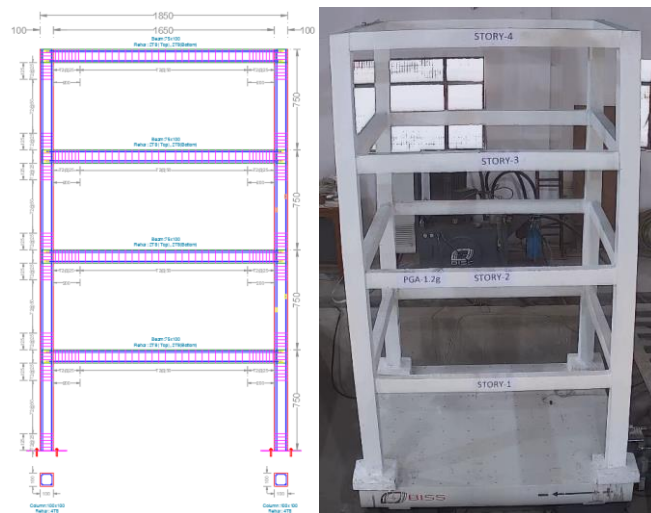


Figure 1. Reinforcement elevation (left) and experimental specimen (right), retained from the author-supplied manuscript for the previously reported programme [1].

The steel-strength units have been normalised to MPa (N/mm²); the original kN/mm² notation is inconsistent with the stated reinforcement strengths. The 25 MPa concrete value is retained as a seven-day result and is not reclassified as a 28-day characteristic strength.

3. Excitation, similarity and frequency assessment

The manuscript describes displacement commands derived from a scaled 1940 El Centro north–south acceleration record. Five successive runs were labelled by nominal PGA values of 0.4, 0.8, 1.2, 1.6 and 2.0 g. These labels identify the runs; they must not be substituted for the achieved table peaks. The record does not fully specify time compression, baseline correction, filtering, or controller performance. The stated similarity scheme assumes the same material properties and neglects gravity similitude. With $S = L_{\text{model}}/L_{\text{prototype}} = 0.25$, the listed ratios are length S , time S , frequency $1/S$, acceleration $1/S$ and force S^2 ; stress, strain, elastic modulus and density have unit ratios. These are assumptions of the adopted idealisation. Physical gravity remains present, and the experiment alone does not establish equivalence to prototype gravity loads or earthquake performance.

Free-vibration assessment was performed by pulling the frame with a rope and recording roof acceleration after release. Frequencies were identified from a Fourier transform of the recorded response. The manuscript does not provide sensor specifications, sampling frequency, record length used for the transform, windowing, detrending, peak-selection criteria, repeated releases, or measurement uncertainty. The post-run values are therefore treated as reported estimates, without assigning confidence intervals.

Table 2. Reported excitation and response peaks.

Nominal PGA (g)	Table peak (g)	Roof peak (g)	Roof/table peak ratio
0.40	0.34	1.09	3.206
0.80	0.67	2.03	3.030
1.20	0.90	2.77	3.078

Nominal PGA (g)	Table peak (g)	Roof peak (g)	Roof/table peak ratio
1.60	0.98	2.98	3.041
2.00	1.34	3.77	2.813

The table peaks range from 61.25% to 85.00% of the nominal labels. Roof/table ratios compare maxima that need not occur at the same instant; they are not frequency-response functions or modal participation factors. The complete measured accelerograms would be needed to assess phase, spectral agreement and time-dependent tracking.

3.1 Reported mathematical-model results

The source supplies a model illustration and five frequency values but does not specify the analysis software, element formulations, mass allocation, concrete modulus, cracked-section stiffness, connection idealisation, or boundary conditions. It also does not explain whether the model was updated after each run or calibrated to the measurements. Accordingly, the model values below are transcribed outputs, not results independently reproduced in this revision.

3.2 Comparison measures

For each run, the signed frequency difference is $\Delta f = f_{\text{model}} - f_{\text{measured}}$ and the relative difference is $e = 100\Delta f / f_{\text{measured}}$. The measured period is $T = 1/f_{\text{measured}}$. Across the five pairs, mean absolute error (MAE) is the mean of $|\Delta f|$ and root-mean-square error (RMSE) is the square root of the mean of Δf^2 . These are descriptive measures for successive states of one specimen, not estimates from five independent experiments.

4. Results and interpretation

Table 3. Frequency comparison and calculated measured periods.

Nominal PGA (g)	Measured f (Hz)	Model f (Hz)	Measured T (s)	Difference (%)
0.40	8.10	8.50	0.1235	4.94
0.80	7.94	8.10	0.1259	2.02
1.20	7.69	7.85	0.1300	2.08
1.60	7.69	7.85	0.1300	2.08
2.00	7.44	7.63	0.1344	2.55

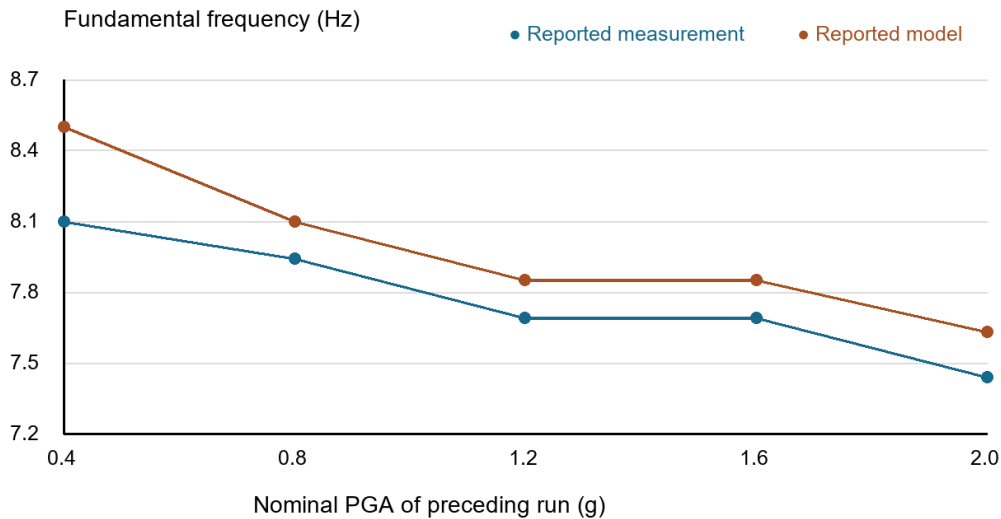


Figure 2. Reported measured and model frequencies after the five runs. Connecting lines indicate the test sequence; no continuous relationship with nominal PGA is inferred.

The measured frequency decreases from 8.10 Hz after the first reported run to 7.44 Hz after the final run, a reduction of 8.15%. The corresponding period increases by 8.87%, from 0.1235 to 0.1344 s. The equal reported values of 7.69 Hz after the 1.2 and 1.6 g runs indicate no resolved change at the stated precision; they do not demonstrate an absence of damage or stiffness change.

The mathematical-model values exceed the measurements for all five pairs. Absolute differences range from 0.16 to 0.40 Hz and relative differences from 2.02% to 4.94%. MAE is 0.214 Hz, RMSE is 0.234 Hz, and mean absolute percentage difference is 2.73%. The largest difference occurs after the first run. These quantities provide a more precise description than an unqualified assertion of good agreement.

A decreasing frequency is consistent with reduced effective stiffness if the participating mass, restraint conditions and identification procedure remain sufficiently stable [2]. The available results do not isolate cracking, support flexibility, or other causes. Moreover, each measurement follows the preceding loading history, so the frequency changes cannot be attributed to nominal PGA alone.

The source prose reports a pre-test frequency of 9.80 Hz, but the accompanying Fourier-spectrum image appears to peak at a lower frequency. That discrepancy requires reconciliation with the original record. The pre-test value has therefore not been used to calculate damage-related frequency loss or stiffness ratios.

5. Scope and limitations

The close frequency values establish numerical proximity for this small set of reported pairs. They do not independently validate a model whose formulation and calibration history are unavailable. Frequency agreement alone cannot verify damping, mode shapes, interstorey drift, nonlinear force–deformation behaviour, or damage localisation. Nor can it establish seismic safety or compliance with a design standard.

The experiment uses one specimen and one earthquake waveform at successive amplitudes. The absence of repeat tests and measurement uncertainty limits generalisation. Achieved table peaks differ from

nominal labels, and measured histories are needed for response-level validation. The gravity-neglected similarity assumptions also limit direct prototype interpretation.

The experimental configuration and tabulated observations overlap with the earlier report [1]. They are presented here with attribution. Any journal submission must identify the relationship to previous conference and journal dissemination and explain which model results or interpretations, if any, have not previously appeared. Editorial changes and a new title do not themselves constitute a new experimental study.

6. Conclusions

The five reported post-run measured frequencies range from 8.10 to 7.44 Hz. The end-to-end decrease is 8.15%, while the corresponding measured period increases from 0.1235 to 0.1344 s.

Reported mathematical-model frequencies are consistently higher than measured values. Relative differences are 2.02–4.94%, with MAE of 0.214 Hz and RMSE of 0.234 Hz.

The differences quantify agreement in frequency only. Missing model specifications, calibration history and measurement uncertainty prevent a claim of independent validation.

The reported experimental observations must be attributed to the earlier programme. A submission decision requires clarification of prior dissemination and the provenance of the model outputs.

References

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