SEISMIC PERFORMANCE EVALUATION REPORT FOR BUILDINGS

Seismic performance evaluation

Sample RC office building

Project	Sample RC office building
Location	Seoul, Republic of Korea (sample site)
Code	KDS 41 17 00 : 2022 (performance-based design, Chapter 15)
Seismic grade	II (IE 1), site S4, category D
Design earthquake	SDS 0.499 g, SD1 0.287 g
Studies	Pushover X (2400 yr), Pushover Y (2400 yr), Time history LS (1000 yr), Time history CP (2400 yr), Time history OP (50 yr)
Solver	OpenSees 3.8.0
File	sample-rc-office.fea

All checks within their limits

Rev.	Date	Description	Prepared	Checked	Approved
0	Nov 10, 2026	First issue			

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1 Design and analysis overview

The building, its structural system, the seismic criteria and the performance objectives the evaluation checks.

1.1 Project overview

Project	Sample RC office building
Location	Seoul, Republic of Korea (sample site)
Use	Office (importance (2), seismic grade II)
Client	—
Structural engineer	FEA Studio sample
Storeys	5 above ground, no basement
Gross floor area	1,080 m^2 (216 m^2 per floor)
Seismic design code	KDS 41 17 00 : 2022 (건축물 내진설계기준), performance-based design (Chapter 15)
Member design codes	ACI, AISC360-LRFD
Analysis software	OpenSees 3.8.0 (nonlinear fibre model, external process) with this application for the model, design, pre- and post-processing
Report date	2026-11-10

A five-storey reinforced concrete intermediate moment frame, 18 m × 12 m in plan (3 × 2 bays of 6 m), storey heights 4.0 m and 3.5 m, generated from the frame-building template with seismic trial reinforcement. A worked example of the seismic performance evaluation by KDS 41 17 00 : 2022.

This is a worked example prepared for review of the evaluation procedure and of the report itself, not the design of a real building. The building is a generated five-storey reinforced concrete frame. Its reinforcement is trial reinforcement (about 0.5 % top and bottom in the beams, 1.5 % in the columns), not the result of a member design. The ground motions are artificial spectrum-compatible pairs generated to the KDS spectrum of each return period (treated as spectrum-matched records, §7.3.4.1(2)②); a project needs recorded motions selected for the site. The site is assumed: seismic zone I, site class S4, seismic grade II.

1.2 Structural system overview

Seismic force-resisting system	3-i. Moment frame: RC intermediate (철근콘크리트 중간모멘트골조): R 5, Ω0 3, Cd 4.5
Storeys modelled	5 above the base at z = 0.00 m
Height hn	18.00 m
Plan	18.0 m × 12.0 m
Seismic weight W	16618 kN (gravity weights of the study's load combination)
Nonlinear members	435 beam elements, 120 column elements

Seismic design category D (Tables 5.2-1, 5.2-2: grade II, SDS 0.499 g, SD1 0.287 g).

Irregularities (§5.3, Tables 5.3-1 and 5.3-2), the stiffness and torsion from the linear static cases EQ X and EQ Y:

Storey	Top z (m)	Stiffness X (kN/mm)	Stiffness Y (kN/mm)	δmax/δavg X	δmax/δavg Y	Strength estimate (kN)	Plan extent (m)
1	4.00	535	567	1.05	1.12	3373	18.0 × 12.0
2	7.50	519	597	1.05	1.12	3855	18.0 × 12.0
3	11.00	513	587	1.05	1.12	3855	18.0 × 12.0
4	14.50	503	568	1.05	1.12	3855	18.0 × 12.0
5	18.00	465	513	1.05	1.11	3855	18.0 × 12.0

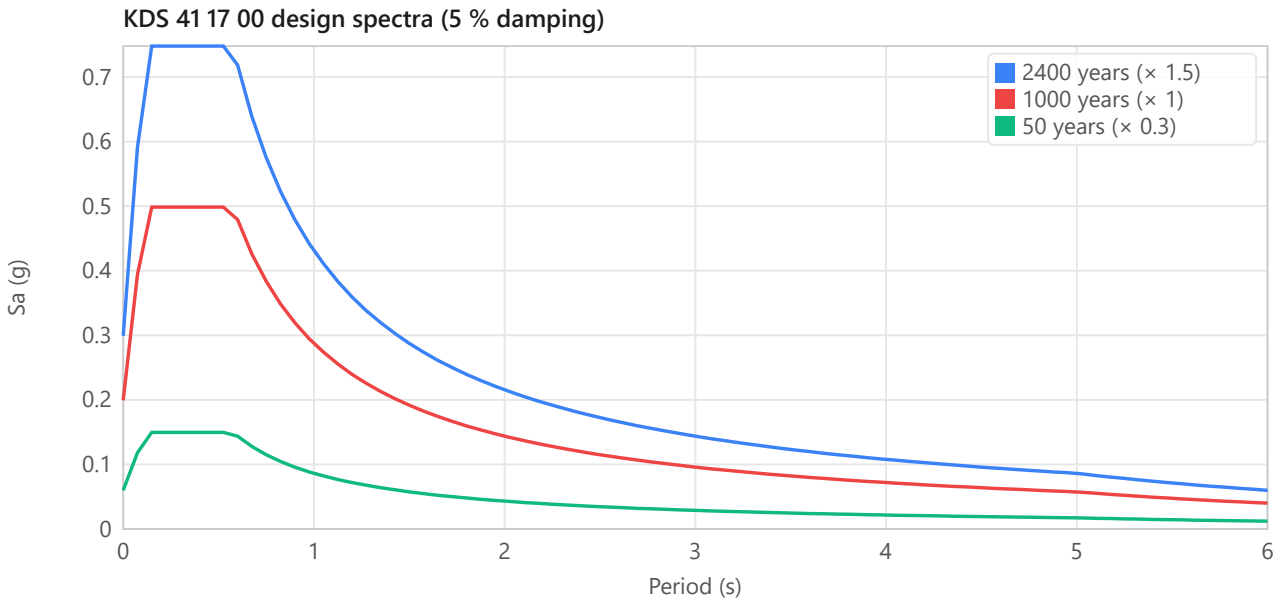
REGULAR None of the irregularities checked was found.

Stiffness: storey shear of the ELF case over the drift of the floor centre. δmax/δavg: the storey drift at the more displaced plan edge over the mean of the two edges, with the accidental eccentricity of the case (H-1 above 1.2). Strength estimate (V-5): the sum over the storey columns of min(Vc + Vs, 2 Mn / hc), Mn without axial load — an index for comparing storeys, not a capacity. Not checked from the results: re-entrant corners H-2, diaphragm discontinuity H-3, out-of-plane offsets H-4, non-parallel systems H-5 and in-plane discontinuity V-4 (they need the floor plans and the load path).

1.3 Seismic design criteria

Code	KDS 41 17 00 : 2022, 건축물 내진설계기준 (revised 2022-10-11)
Seismic grade / IE	II, IE = 1 (Table 2.2-1)
Zone / effective ground acceleration S	zone I, S = 0.220 g — Z·I = 0.11 × 2 (zone I, §3.2)
Site class	S4
Site coefficients	Fa = 1.360, Fv = 1.960 (Tables 4.2-1, 4.2-2, interpolated in S)
Design earthquake (§2.3: 2/3 of the 2400-year MCE)	SDS = 0.499 g, SD1 = 0.287 g, TL = 5 s
Seismic design category	D (Tables 5.2-1, 5.2-2)
Force-resisting system	3-i. Moment frame: RC intermediate (철근콘크리트 중간모멘트골조): R 5, Ω0 3, Cd 4.5

$$
\begin{aligned}
S_{\mathrm{DS}} &= \frac{S \cdot 2.5 \cdot F_a \cdot 2}{3} \\
&= \frac{0.220 \cdot 2.5 \cdot 1.360 \cdot 2}{3} \\
&= 0.499g \\
S_{\mathrm{D1}} &= \frac{S \cdot F_v \cdot 2}{3} \\
&= \frac{0.220 \cdot 1.960 \cdot 2}{3} \\
&= 0.287g \\
T_0 &= \frac{0.2 \cdot S_{\mathrm{D1}}}{S_{\mathrm{DS}}} \\
&= 0.115\ \mathrm{s} \\
T_s &= \frac{S_{\mathrm{D1}}}{S_{\mathrm{DS}}} \\
&= 0.576\ \mathrm{s}
\end{aligned}
$$



Spectra of the return periods of the objectives: the design earthquake (Fig. 4.2-1) times the factors of §15.3.2

The zone factors Z (KDS 17 10 00 Table 4.2-2) are not in KDS 41 17 00 itself; check them, or enter S from the national hazard map. Site-specific response analysis (§4.2.3 ③) is not used.

1.4 Performance objectives

The objectives of a performance-based design of grade II (Table 15.3-1); at least two must be met (§15.3.1). A building level asks for the structural level of Table 2.4-1 (habitable = IO, life safety = LS, collapse prevention = CP).

Return period	Building level	Structural level	SDS / SD1 (g)	Storey drift limit	Members (ASCE 41)	Study
2400 years	Collapse prevention (붕괴방지)	CP	0.748 / 0.431	3.00 %	CP	Pushover X (2400 yr), Pushover Y (2400 yr), Time history CP (2400 yr)
1000 years	Life safety (인명보호)	LS	0.499 / 0.287	2.00 %	LS	Time history LS (1000 yr)

Return period	Building level	Structural level	SDS / SD1 (g)	Storey drift limit	Members (ASCE 41)	Study
50 years	Operational (기능수행)	IO	0.150 / 0.086	0.50 %	IO (may be omitted)	Time history OP (50 yr)

Drift limits: operational 1.0 % (special) or 0.5 % (I, II) (§15.4(2)); life safety Table 8.2-1 (0.010 / 0.015 / 0.020 hsx), which the nonlinear response must meet (§7.3.4.3, §8.2.3); collapse prevention at the MCE 3 % / IE (§15.4(3)). Grade I collapse prevention: member limits ÷ 1.2 (Table 15.3-1 note 1). KDS 41 17 00 gives no member deformation limits of its own: the ASCE 41-17 acceptance criteria are used.

2 Structural analysis model

From the BIM model to the nonlinear OpenSees model: geometry, materials, sections, nonlinear modelling, supports and loads.

2.1 BIM model

Source file	5-storey concrete building, 18 × 12 m.ifc
Authoring application	FEA Template
Schema	IFC4
Project	5-storey concrete building, 18 × 12 m
Imported	2026-11-10 04:51
Length unit	m
Building storeys	—
Joints	eccentric member ends joined by rigid links
Rigid links	0

Element role	Elements
column	60
beam	225
slab	90
footing	12

Every analysis object keeps the GlobalId of the BIM element it was made from, so results and reinforcement are written back to it. 12 elements carry idealisation notes (BIM sheet).

2.2 OpenSees analysis model

A three-dimensional model of the whole structure (KDS 41 17 00 §1.6(4), §7.3.2): the members with bars from the design as fibre force-based elements, the walls as MVLEM_3D panels, rigid diaphragms where the floors are, the gravity loads first. The structural model is generated from the design model by this application; nothing is entered in OpenSees by hand.

Frame

Model	422 nodes, 555 line elements, 270 shell elements (2,532 degrees of freedom)
Fibre elements	555 of the line elements (16 sections)
Height	18.00 m to the control node 401
Direction	global X

The studies below ran on the same model as solved; their own sizes are in their chapters.

Solver and settings

Solver	OpenSees 3.8.0 — run as the user's own installation (external process), not shipped with this software

Elements	forceBeamColumn (mixedBeamColumn in the time history: the same force-based interpolation) with fibre sections (Lobatto, 5 points), rigid joint offsets, P-Δ geometric transformation for columns
Solution	Newton with line search / Krylov–Newton fallbacks, norm displacement increment test 1e-8, step halving on non-convergence
Created	Pushover X (2400 yr): 10/11/2026, 13:22:32, 74.9 s; Pushover Y (2400 yr): 10/11/2026, 12:01:50, 219.8 s; Time history LS (1000 yr): 10/11/2026, 15:55:29, 2062.0 s; Time history CP (2400 yr): 10/11/2026, 15:59:34, 2192.9 s; Time history OP (50 yr): 10/11/2026, 15:51:05, 1907.8 s

OpenSees (Open System for Earthquake Engineering Simulation, UC Berkeley / PEER) is separate software under its own licence; the version is the one the solver reported for this run.

2.3 Material properties and section definitions

2.3.1 Materials

Concrete (cover)	f′c = 28.0 MPa — Concrete02, no tension unless noted
Concrete (confined core)	Kent–Park / Mander: f′cc 29.9–31.5 MPa, εcu 13.0–19.6 ‰ (Priestley)
Reinforcement / steel	fy = 420 MPa, Es = 200 GPa — Steel02 (Giuffré–Menegotto–Pinto)

Strengths are expected (mean) values as entered for the design; ASCE 41 asks for expected strengths in nonlinear procedures — check the factors applied.

2.3.2 Fibre sections of the RC members

Section	Member	b × h (mm)	Bars	As (mm²)	ρ (%)	Ties	f′cc (MPa)	Vn,y / Vn,z (kN)	Elements
B1 400×700 T 850×200	beam	400 × 700 (flange 850 × 200)	13	3725	1.33	Av 142 mm² @ 150 mm	29.9	350 / 485	90
B2 400×700	beam	400 × 700	13	3725	1.33	Av 142 mm² @ 150 mm	29.9	350 / 485	300
B4 400×700 T 1750×200	beam	400 × 700 (flange 1750 × 200)	13	3725	1.33	Av 142 mm² @ 150 mm	29.9	350 / 485	45
C1 storey 1	column	600 × 600	12	6080	1.69	Av 213 mm² @ 100 mm	31.5	771 / 771	120

Each section is meshed into unconfined cover, confined core and bar fibres; the members are force-based elements with Lobatto integration (or plastic hinge integration where chosen). Vn = Vc + Vs of ACI 318 from the ties.

2.4 Nonlinear modelling

Item	Model
Beams and columns	forceBeamColumn (mixedBeamColumn in the time history) with fibre sections, 5 Lobatto points; distributed plasticity
Concrete	Concrete02 (Kent–Park, linear tension softening); confined core by Mander from the ties, εcu by Priestley
Reinforcement / steel	Steel02 (Giuffré–Menegotto–Pinto), expected strengths as entered
Plastic rotation	θp = (φ − φy)·Lh at each member end (Lh = L/20, the end Lobatto weight), plus bar slip beyond yield where modelled

Item	Model
Shear	Vn = Vc + Vs from the ties checked at every step (force-controlled); the pushover stops at the first shear failure when set
Joints	rigid zones of 0.25 × the half-depth; joint shear checked by ASCE 41-17 Eq. 10-4
Walls	MVLEM_3D panels (vertical Concrete02 / Steel02 fibres, elastic shear), rows merged into full-width panels
Geometric nonlinearity	P-Δ transformation of the columns (KDS 41 17 00 §7.3.2 ⑤)
Damping	Rayleigh 5.0 % at modes 2 and 5
Mass	lumped at the nodes from the gravity weights of the study's load combination
Integration / solution	Newmark average acceleration; Newton with line search and Krylov–Newton fall-backs, step halving down to 1/64

KDS 41 17 00 §1.6: the mass and the initial stiffness are not underestimated (gross sections crack in the fibres as they are loaded), the post-yield stiffness is not overestimated (Steel02 hardening as entered, softening concrete), and the model is three-dimensional. §9.8.1: the concrete members crack in the model itself rather than by fixed stiffness modifiers.

2.5 Boundary conditions and loading

Supports: 12 nodes fixed (fixed base: KDS 41 17 00 §7.2.9(1) allows a fixed base or springs).

Study	Combination
Pushover X (2400 yr)	1.00 Dead + 1.00 Superimposed dead + 0.25 Live
Pushover Y (2400 yr)	1.00 Dead + 1.00 Superimposed dead + 0.25 Live
Time history LS (1000 yr)	1.00 Dead + 1.00 Superimposed dead + 0.25 Live
Time history CP (2400 yr)	1.00 Dead + 1.00 Superimposed dead + 0.25 Live
Time history OP (50 yr)	1.00 Dead + 1.00 Superimposed dead + 0.25 Live

The gravity loads are applied first (load control, then held constant); their downward weights are also the seismic weight of the lateral pattern, the capacity spectrum and the mass of the time history.

Storey	Top z (m)	Weight (kN)
1	4.00	3582
2	7.50	3366
3	11.00	3366
4	14.50	3366
5	18.00	2937

Seismic weight per storey: the gravity weights of the study's combination at the nodes of each floor (KDS 41 17 00 §7.2.1: dead load, 25 % of storage live load, partitions, permanent equipment, 20 % of snow above 1.5 kN/m^2; check the combination).

3 Seismic analysis

The modes, the linear check, the pushover and time history analyses and the records they used.

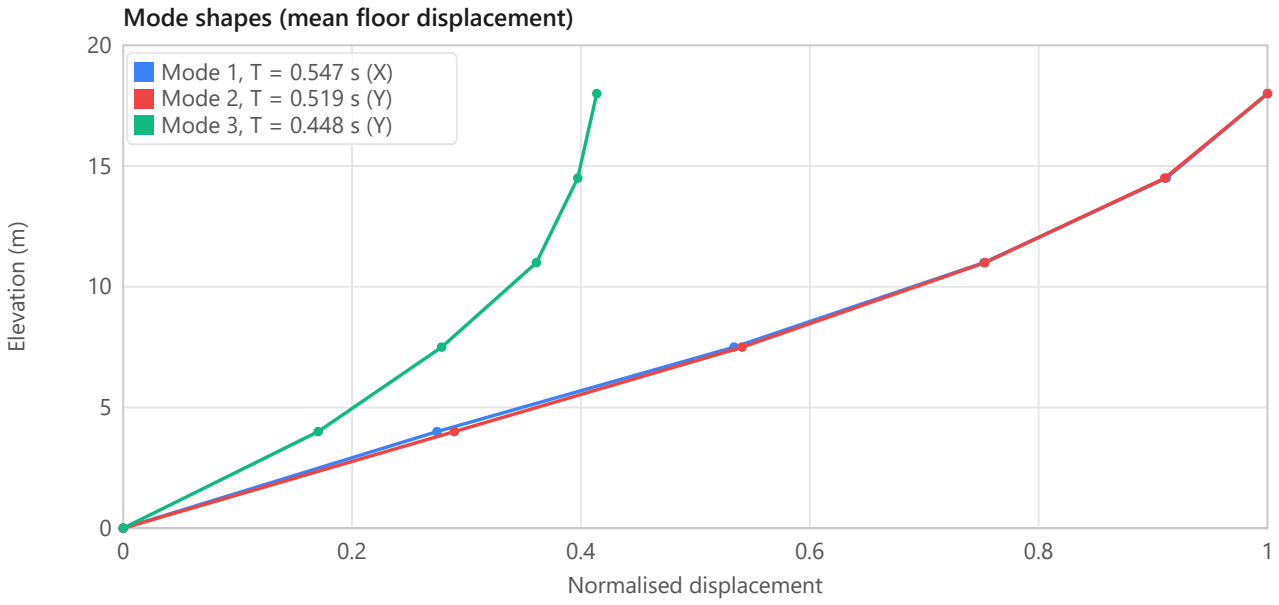
3.1 Eigenvalue analysis and mode shapes

Modes of the nonlinear model under its gravity loads (OpenSees, before the record), mass 1694.0 t:

Mode	T (s)	Mass along	Mass across	Σ along	Σ across
1	0.558	0.0 %	86.7 %	0.0 %	86.7 %
2	0.515	87.8 %	0.0 %	87.8 %	86.7 %
3	0.461	0.0 %	0.0 %	87.8 %	86.7 %
4	0.180	0.0 %	9.3 %	87.8 %	96.0 %
5	0.167	8.8 %	0.0 %	96.6 %	96.0 %
6	0.149	0.0 %	0.0 %	96.6 %	96.0 %
7	0.102	0.0 %	2.6 %	96.6 %	98.6 %
8	0.096	2.3 %	0.0 %	98.8 %	98.6 %
9	0.085	0.0 %	0.0 %	98.8 %	98.6 %
10	0.079	0.0 %	0.0 %	98.8 %	98.6 %
11	0.070	0.0 %	0.9 %	98.8 %	99.4 %
12	0.069	0.2 %	0.0 %	99.1 %	99.4 %

Mass participation of the modes listed reaches 90 % in each direction (KDS 41 17 00 §7.3.3.1).

Linear modal analysis of the design model (Modal 1, Project Chrono 10.0.0): T1 = 0.547 s, T2 = 0.519 s, T3 = 0.448 s.



First mode shapes of the linear model, mean horizontal floor displacement in the dominant direction

3.2 Linear analysis verification

KDS 41 17 00 §15.5 ④ asks for the period, the largest base shear, the share and failure mode of the main lateral elements, the drift profile and the roof displacement of the nonlinear analyses to be compared with a linear analysis, and the differences explained; §15.6 that the design base shear be at least 75 % of the equivalent static one.

$$
\begin{aligned}
W &= 16618\ \text{kN} \\
T_a &= C_t \cdot h_n^x \\
&= 0.628\ \text{s} \\
T &= \min(T_1, C_u \cdot T_a) \\
&= 0.515\ \text{s} \\
C_s &= \frac{S_{\text{DS}}}{\frac{R}{I_E}} \le \frac{S_{\text{D1}}}{\left(\frac{R}{I_E}\right) \cdot T},\ \ge 0.044 \cdot S_{\text{DS}} \cdot I_E \ge 0.01 \rightarrow 0.0997 (R = 5, I_E = 1) \\
V &= C_s \cdot W \\
&= 1657\ \text{kN}
\end{aligned}
$$

Quantity	Analysis	Value
Period	Ta = 0.628 s, Cu·Ta = 0.887 s (Table 7.2-1)	T used = 0.515 s
Period	OpenSees modes (nonlinear model, uncracked)	T1 = 0.515 s
Period	pushover initial stiffness (Pushover X (2400 yr))	Ti = 0.517 s
Period	pushover initial stiffness (Pushover Y (2400 yr))	Ti = 0.560 s
Base shear	equivalent static V = Cs·W (§7.2.1, plateau governs)	1657 kN (Cs = 0.0997)
Base shear	pushover yield Vy (Pushover X (2400 yr))	4273 kN = 2.58 V
Base shear	pushover peak (Pushover X (2400 yr))	5013 kN = 3.02 V (Ω0 = 3)
Base shear	pushover yield Vy (Pushover Y (2400 yr))	4116 kN = 2.48 V
Base shear	pushover peak (Pushover Y (2400 yr))	4843 kN = 2.92 V (Ω0 = 3)
Base shear	time history peak (Time history LS (1000 yr))	4562 kN = 2.75 V
Base shear	time history peak (Time history CP (2400 yr))	4517 kN = 2.73 V
Base shear	time history peak (Time history OP (50 yr))	2114 kN = 1.28 V
Base shear	linear static Static 1 — Wind X	X 238 kN, Y 0 kN
Base shear	linear static Static 1 — Wind Y	X 0 kN, Y 389 kN
Base shear	linear static Static 1 — EQ X	X 1229 kN, Y 0 kN
Base shear	linear static Static 1 — EQ Y	X 0 kN, Y 1229 kN
Failure mode	pushover (Pushover X (2400 yr))	first yield element 13; at the end 2 beam and 32 column hinge ends yielded
Failure mode	pushover (Pushover Y (2400 yr))	first yield element 3; at the end 2 beam and 32 column hinge ends yielded
Roof drift	Pushover X (2400 yr)	0.54 %

Quantity	Analysis	Value
Roof drift	Pushover Y (2400 yr)	0.60 %
Largest storey drift of any run	Time history LS (1000 yr)	1.10 %
Largest storey drift of any run	Time history CP (2400 yr)	2.17 %
Largest storey drift of any run	Time history OP (50 yr)	0.14 %

$$\begin{aligned} 0.75 \cdot V &= 0.75 \cdot 1657 \\ &= 1243\ \text{kN} \leq V_y \\ &= 4116\ \text{kN} \end{aligned}$$

STRENGTH ≥ 75 % OF THE STATIC BASE SHEAR

A linear static or spectrum study of the design model, loaded with these results, adds its base shears here. Differences to expect: the nonlinear model's first period is that of uncracked sections (shorter than a cracked-section linear model), and its strength includes the overstrength that Ω0 stands for.

3.3 Pushover analysis

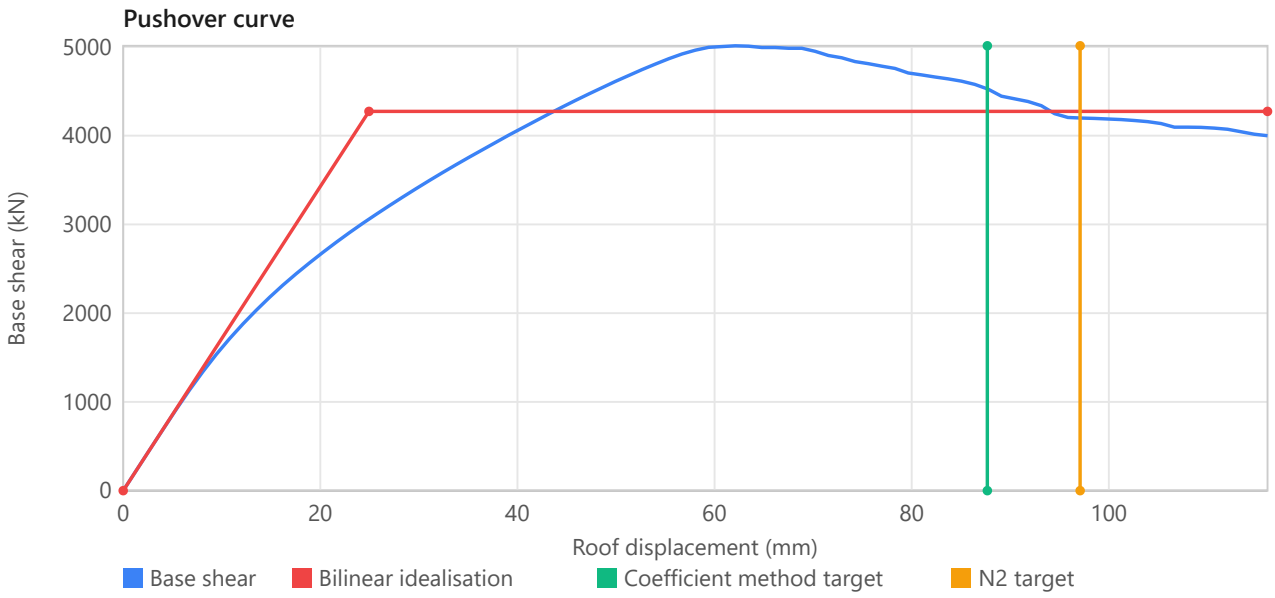
3.3.1 Pushover — Pushover X (2400 yr)

Monotonic frame pushover with fibre sections: gravity loads first, then the triangular lateral pattern under displacement control of the roof.

3.3.1.1 Settings

Direction	global X, control node 401
Lateral pattern	triangular: Fi ∝ wi·zi
Loading	monotonic to 3.0 % roof drift
Steps	400 (87 converged points)
Member shear check	on, the push stops at the first shear failure
Rigid joint zone	0.25 of the half-depth
Bar slip / hinge integration	no bar slip; Lobatto fibre elements (θp over L/20)

3.3.1.2 Capacity curve



Capacity curve with the bilinear idealisation and the target displacements

Point	Roof displacement (mm)	Drift	Base shear (kN)
First bar yield (bar strain ≥ fy/Es), element 13	43	0.24 %	4246
Bilinear idealisation yield point	25	0.14 %	4273
First core crushing (strain ≥ εcu), element 21	80	0.44 %	4705
Peak	62	0.34 %	5013
Ultimate (strength down to 80 % of the peak: our stop rule)	116	0.65 %	4000

The events are threshold checks on the analysis, not outcomes it reports: yield when a watched bar of a hinge-end section (the extreme bars, a sample of the section) reaches fy/Es, crushing when a watched core-corner fibre reaches εcu, shear when a member end shear reaches Vc + Vs. The push ends where the strength has fallen to 80 % of the peak (our stop rule, after the ASCE 41 / FEMA 356 ultimate point).

3.3.1.3 *Bilinear idealisation*

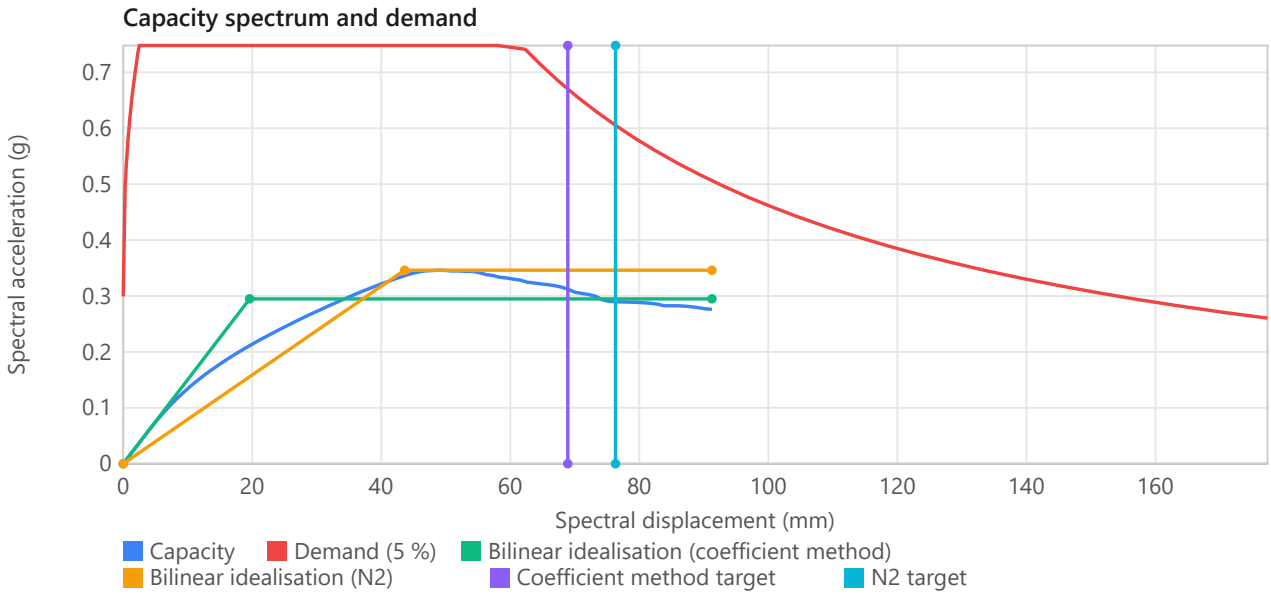
Elastic–perfectly plastic idealisation computed with the run: the initial stiffness (first step) and equal energy up to the ultimate point (the same area under both curves). The coefficient method below takes Vy from it but its own effective stiffness, the secant through the curve at 0.6 Vy; the N2 method idealises the curve its own way (Fy* the largest force).

$$
\begin{aligned}
V_y &= 4273\ \text{kN}, \Delta_y = 24.9\ \text{mm} \\
K_y &= \frac{V_y}{\Delta_y} \\
&= \frac{4273\ \text{kN}}{24.9\ \text{mm}} \\
&= 171387\ \text{kN/m} \\
\mu &= \frac{\Delta_u}{\Delta_y} \\
&= \frac{116.1}{24.9} \\
&= 4.66 \\
\frac{V_y}{W} &= \frac{4273}{16618} \\
&= 0.257 \\
\Omega &= \frac{Vm_{\text{ax}}}{V_y} \\
&= \frac{5013}{4273} \\
&= 1.17
\end{aligned}
$$

3.3.1.4 *Capacity spectrum*

The curve converted to the equivalent single-degree-of-freedom system in acceleration–displacement format with the nodal weights and the elastic pushed shape φ (roof = 1): the lateral displacements the first push step adds to the gravity state.

$$
\begin{aligned}
\Gamma &= \sum (w \cdot \varphi) \,/\, \sum (w \cdot \varphi^2) \\
&= 1.272 \\
\alpha &= \frac{(\sum w \cdot \varphi)^2}{W \cdot \sum (w \cdot \varphi^2)} \\
&= 0.872 \\
W &= 16618\ \text{kN} \\
S_d &= \frac{\Delta_{\text{roof}}}{\Gamma} \\
S_a &= \frac{V}{\alpha \cdot W} \\
T_i &= 0.517\ \text{s}
\end{aligned}
$$



Capacity spectrum against the 5 % damped design demand (ADRS)

3.3.1.5 Performance point

Method	Target roof displacement (mm)	Roof drift	Within the push
ASCE 41-17 coefficient method	87.7	0.49 %	yes
N2	97.1	0.54 %	yes

3.3.1.5.1 *Coefficient method (ASCE 41-17 7.4.3.3)*

$$K_i = \frac{V_1}{\Delta_1}$$
$$= \frac{231.4\ \text{kN}}{1.35\ \text{mm}}$$
$$= 171387\ \text{kN/m}$$
$$\Delta_e = 18.9\ \text{mm}$$

the roof displacement where the curve reaches 0.6 Vy

$$K_e = \frac{0.6 \cdot V_y}{\Delta_e}$$
$$= \frac{0.6 \cdot 4273\ \text{kN}}{18.9\ \text{mm}}$$
$$= 135890\ \text{kN/m}$$
$$T_i = 2\pi \cdot \sqrt{\frac{S_{\text{d1}}}{S_{\text{a1}} \cdot g}}$$
$$= 0.517\ \text{s}$$
$$T_e = T_i \cdot \sqrt{\frac{K_i}{K_e}}$$
$$= 0.517 \cdot \sqrt{\frac{171387}{135890}}$$
$$= 0.581\ \text{s}$$
$$S_a = \frac{S_{\text{D1}}}{T_e}$$
$$= \frac{0.431}{0.581}$$
$$= 0.742g$$
$$W = 16618\ \text{kN}$$
$$C_m = 1.00(g_{\text{iven}})$$
$$R = \frac{S_a}{\frac{V_y}{W}} \cdot C_m$$
$$= \frac{0.742}{\frac{4273}{16618}} \cdot 1.00$$
$$= 2.887$$
$$C_0 = \Gamma$$
$$= 1.272$$
$$\mu m_{\text{ax}} = \frac{\Delta_d}{\Delta_y} + |\alpha_e|\ \hat{}\ - \frac{h}{4}$$
$$= 7.71(\text{Eq}\ .7 - 32)$$

R ≤ μmax: the NSP is permitted

$$C_1 = 1 + \frac{R-1}{a \cdot T_e^2}$$
$$= 1 + \frac{2.887 - 1}{60 \cdot 0.581^2}$$
$$= 1.093 \ \text{(a = 60 for site class D)}$$

$$C_2 = 1 + \frac{\left(\frac{R-1}{T_e}\right)^2}{800}$$
$$= 1 + \frac{\left(\frac{2.887-1}{0.581}\right)^2}{800}$$
$$= 1.013$$

$$\delta_t = \frac{C_0 \cdot C_1 \cdot C_2 \cdot S_a \cdot T_e^2 \cdot g}{4\pi^2}$$
$$= \frac{1.272 \cdot 1.093 \cdot 1.013 \cdot 0.742 \cdot 0.581^2 \cdot 9.807}{4\pi^2}$$
$$= 87.7 \text{ mm}$$

g = 9.807 m/s²

$$\frac{\delta_t}{H} = \frac{87.7 \text{ mm}}{18.00 \text{ m}}$$
$$= 0.49\%$$

Te, C0, C1, C2, Cm and μmax are checked against ASCE/SEI 41-17 §7.4.3 (Eqs. 7-27 to 7-33, Table 7-4); αPΔ in μmax is approximated by −W/(H·Ke).

3.3.1.5.2 N2 method (EN 1998-1 Annex B)

$$\Gamma = \sum (m \cdot \varphi) \,/\, \sum (m \cdot \varphi^2)$$
$$= 1.272$$
$$F_y^* = 0.3460g$$

the largest spectral acceleration of the equivalent system

$$d_m^* = \frac{\Delta_u}{\Gamma}$$
$$= 91.3 \text{ mm}$$
$$E_m^* = 24.025g \cdot \text{mm}$$

the area under the curve to dm*

$$d_y^* = 2 \cdot \left(d_m^* - \frac{E_m^*}{F_y^*} \right)$$
$$= 2 \cdot \left(91.3 - \frac{24.025}{0.3460} \right)$$
$$= 43.62 \text{ mm}$$
$$T^* = 2\pi \cdot \sqrt{\frac{d_y^*}{F_y^* \cdot g}}$$
$$= 2\pi \cdot \sqrt{\frac{0.04362}{0.3460 \cdot 9.807}}$$
$$= 0.712 \text{ s}$$
$$S_e = 0.605g$$

the design spectrum at T*

$$S_{\text{de}} = S_e \cdot g \cdot \left(\frac{T^*}{2\pi} \right)^2$$
$$= 0.605 \cdot 9.807 \cdot \left(\frac{0.712}{2\pi} \right)^2$$
$$= 76.3 \text{ mm}$$
$$q_u = \frac{S_e}{F_y^*}$$
$$= \frac{0.605}{0.3460}$$
$$= 1.749$$
$$T_c = \frac{S_{\text{D1}}}{S_{\text{DS}}}$$
$$= 0.576 \text{ s}$$
$$d_t^* = S_{\text{de}}$$
$$= 76.3 \text{ mm}$$

T* ≥ Tc, equal displacements

$$
\begin{aligned}
d_t &= \Gamma \cdot d_t^* \\
&= 1.272 \cdot 76.3\ \text{mm} \\
&= 97.1\ \text{mm} \\
\frac{d_t}{H} &= 0.54\%
\end{aligned}
$$

With the ASCE 7 spectrum shape and Tc = SD1/SDS; checked against a published worked example.

3.3.1.6 *Hinge acceptance at the targets*

Plastic rotation θp = (φ − φy)·Lh at each yielded member end (Lh = L/20, the end Lobatto weight, or the hinge length with hinge integration; plus bar slip beyond yield) against the ASCE 41-17 Table 10-7 / 10-8 limits interpolated on the member parameters.

$$
\theta_p = (\varphi - \varphi_y) \cdot L_h
$$

Table parameters: RC beams (ρ − ρ′)/ρbal and V/(bw d √f′c) in psi units, conforming ties s ≤ d/3; RC columns P/(Ag f′c), the transverse ratio ρt = Av/(b s) (≤ 0.0175) and VyE/VColOE (≥ 0.2), IO 0.15a ≤ 0.005, LS 0.5b, CP 0.7b with b at P/(Ag f′c) ≥ 0.1 (Table 10-9 for circular columns with hoops); steel θy = Mpe L/(6 E I), columns Mpce L/(6 τb E I) at the gravity axial force.

The limits and coefficients are checked against ASCE/SEI 41-17: steel Table 9-7.1 and Eqs. 9-1 to 9-7; RC beams Table 10-7; RC columns Tables 10-8 and 10-9 with their notes and Eq. 10-3; RC walls Table 10-19; joints Table 10-12 and Eq. 10-4; target displacement Eqs. 7-27 to 7-33 and Tables 7-4 and 7-5. The FEMA 356 coefficients and N2 are checked against published worked examples. Engineering judgement: a roof joint with beams on two opposite faces is classed as exterior; the wall check assumes unconfined boundaries, low axial load and high shear because the piers' forces are not in the results.

3.3.1.6.1 *At the coefficient method target (step 65, 88 mm)*

WORST LS 330 hinges: 294 elastic, 14 IO, 22 LS, 0 CP, 0 beyond CP.

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
21	i	column	C1 storey 1	10.5	3.2	17.7	24.8	0.42	LS
9	i	column	C1 storey 1	9.7	3.4	19.7	27.6	0.35	LS
15	i	column	C1 storey 1	9.8	3.5	20.2	28.3	0.35	LS
19	i	column	C1 storey 1	10.0	3.8	21.5	30.1	0.33	LS
23	i	column	C1 storey 1	10.0	3.8	21.5	30.1	0.33	LS
10	j	column	C1 storey 1	8.7	3.2	19.8	27.8	0.31	LS
16	j	column	C1 storey 1	8.8	3.3	20.3	28.5	0.31	LS
22	j	column	C1 storey 1	7.5	3.0	17.8	24.9	0.30	LS
11	i	column	C1 storey 1	9.5	4.0	24.3	34.1	0.28	LS
7	i	column	C1 storey 1	9.5	4.0	24.3	34.1	0.28	LS
13	i	column	C1 storey 1	9.7	4.1	25.1	35.2	0.28	LS
17	i	column	C1 storey 1	9.7	4.1	25.1	35.2	0.27	LS
12	j	column	C1 storey 1	7.8	3.8	24.5	34.3	0.23	LS
8	j	column	C1 storey 1	7.8	3.8	24.5	34.3	0.23	LS
24	j	column	C1 storey 1	6.8	3.5	21.6	30.3	0.22	LS
20	j	column	C1 storey 1	6.8	3.5	21.6	30.3	0.22	LS

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
18	j	column	C1 storey 1	7.8	3.9	25.3	35.4	0.22	LS
14	j	column	C1 storey 1	7.8	3.9	25.3	35.4	0.22	LS
3	i	column	C1 storey 1	8.6	5.0	29.0	40.6	0.21	LS
5	i	column	C1 storey 1	8.0	5.0	29.0	40.6	0.20	LS
1	i	column	C1 storey 1	8.0	5.0	29.0	40.6	0.20	LS
4	j	column	C1 storey 1	5.0	4.9	29.0	40.6	0.12	LS
6	j	column	C1 storey 1	4.1	5.0	29.0	40.6	0.10	IO
2	j	column	C1 storey 1	4.1	5.0	29.0	40.6	0.10	IO
127	j	column	C1 storey 1	0.2	3.6	22.6	31.6	0.01	IO
121	j	column	C1 storey 1	0.2	3.6	23.4	32.8	0.01	IO
120	i	column	C1 storey 1	0.2	3.6	23.3	32.7	0.01	IO
126	i	column	C1 storey 1	0.2	3.5	22.5	31.5	0.01	IO
25	i	beam	B1 400×700 T 850×200	0.2	8.8	23.8	45.3	0.00	IO
79	i	beam	B1 400×700 T 850×200	0.2	8.8	23.8	45.3	0.00	IO

6 more yielded hinges are within IO or better.

Joint shear at this step: 0 of 60 joints over capacity, worst 0.46 at node 20.

3.3.1.6.2 At the N2 target (step 72, 97 mm)

WORST LS 330 hinges: 296 elastic, 10 IO, 24 LS, 0 CP, 0 beyond CP.

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
21	i	column	C1 storey 1	13.3	3.2	17.7	24.8	0.54	LS
9	i	column	C1 storey 1	13.0	3.4	19.7	27.6	0.47	LS
15	i	column	C1 storey 1	12.8	3.5	20.2	28.3	0.45	LS
22	j	column	C1 storey 1	11.3	3.0	17.8	24.9	0.45	LS
19	i	column	C1 storey 1	13.3	3.8	21.5	30.1	0.44	LS
23	i	column	C1 storey 1	13.3	3.8	21.5	30.1	0.44	LS
16	j	column	C1 storey 1	12.3	3.3	20.3	28.5	0.43	LS
10	j	column	C1 storey 1	11.5	3.2	19.8	27.8	0.41	LS
11	i	column	C1 storey 1	12.4	4.0	24.3	34.1	0.36	LS
7	i	column	C1 storey 1	12.4	4.0	24.3	34.1	0.36	LS
13	i	column	C1 storey 1	12.5	4.1	25.1	35.2	0.36	LS
17	i	column	C1 storey 1	12.5	4.1	25.1	35.2	0.36	LS
24	j	column	C1 storey 1	9.9	3.5	21.6	30.3	0.33	LS
20	j	column	C1 storey 1	9.9	3.5	21.6	30.3	0.33	LS
12	j	column	C1 storey 1	11.1	3.8	24.5	34.3	0.32	LS
8	j	column	C1 storey 1	11.1	3.8	24.5	34.3	0.32	LS
14	j	column	C1 storey 1	11.3	3.9	25.3	35.4	0.32	LS

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
18	j	column	C1 storey 1	11.3	3.9	25.3	35.4	0.32	LS
3	i	column	C1 storey 1	11.2	5.0	29.0	40.6	0.28	LS
5	i	column	C1 storey 1	10.5	5.0	29.0	40.6	0.26	LS
1	i	column	C1 storey 1	10.5	5.0	29.0	40.6	0.26	LS
4	j	column	C1 storey 1	8.1	4.9	29.0	40.6	0.20	LS
6	j	column	C1 storey 1	7.1	5.0	29.0	40.6	0.18	LS
2	j	column	C1 storey 1	7.1	5.0	29.0	40.6	0.18	LS
127	j	column	C1 storey 1	0.2	3.6	22.6	31.6	0.01	IO
121	j	column	C1 storey 1	0.2	3.6	23.4	32.8	0.01	IO
126	i	column	C1 storey 1	0.2	3.5	22.5	31.5	0.01	IO
120	i	column	C1 storey 1	0.2	3.6	23.3	32.7	0.00	IO
25	i	beam	B1 400×700 T 850×200	0.2	8.8	23.8	45.3	0.00	IO
79	i	beam	B1 400×700 T 850×200	0.2	8.8	23.8	45.3	0.00	IO

4 more yielded hinges are within IO or better.

Joint shear at this step: 0 of 60 joints over capacity, worst 0.46 at node 20.

3.3.1.7 Member checks

3.3.1.7.1 Shear

Member shear at the larger target (step 72).

NO SHEAR FAILURE No member reached its shear capacity Vc + Vs.

Element	End	Section	V max (kN)	Vn (kN)	V / Vn	Status
126	i	C1 storey 1	551	771	0.72	OK
127	j	C1 storey 1	549	771	0.71	OK
120	i	C1 storey 1	543	771	0.70	OK
121	j	C1 storey 1	540	771	0.70	OK
60	j	B4 400×700 T 1750×200	239	350	0.68	OK
21	i	C1 storey 1	515	771	0.67	OK
22	j	C1 storey 1	515	771	0.67	OK
9	i	C1 storey 1	512	771	0.66	OK
10	j	C1 storey 1	512	771	0.66	OK
15	i	C1 storey 1	503	771	0.65	OK
16	j	C1 storey 1	502	771	0.65	OK
132	i	C1 storey 1	492	771	0.64	OK
133	j	C1 storey 1	489	771	0.63	OK
23	i	C1 storey 1	471	771	0.61	OK
19	i	C1 storey 1	471	771	0.61	OK

V max is the resultant section shear; Vn the smaller of the two local capacities (conservative). The run itself compares each component with its own capacity (the events above).

3.3.1.7.2 Beam–column joints

Joint shear at the larger target (step 72).

ALL WITHIN CAPACITY 60 beam–column joints; worst node 20 at Vj/Vn = 0.46.

Node	Joint	γ	Vj (kN)	Vn (kN)	Vj / Vn	Status
20	exterior with transverse beams	15	1096	2372	0.46	OK
24	exterior with transverse beams	15	1096	2372	0.46	OK
22	interior with transverse beams	20	1203	3162	0.38	OK
8	interior with transverse beams	20	1189	3162	0.38	OK
12	interior with transverse beams	20	1189	3162	0.38	OK
10	interior with transverse beams	20	1171	3162	0.37	OK
14	interior with transverse beams	20	1151	3162	0.36	OK
18	interior with transverse beams	20	1151	3162	0.36	OK
50	exterior with transverse beams	15	833	2372	0.35	OK
52	exterior with transverse beams	15	833	2372	0.35	OK
16	interior with transverse beams	20	1079	3162	0.34	OK
51	interior with transverse beams	20	999	3162	0.32	OK
44	interior with transverse beams	20	996	3162	0.31	OK
46	interior with transverse beams	20	996	3162	0.31	OK
47	interior with transverse beams	20	983	3162	0.31	OK

$$V_j = \sum \frac{M}{0.9 \cdot d} - V_{\text{col}}$$

$$V_n = 0.083 \cdot \gamma \cdot \sqrt{f_c'} \cdot A_j$$ (ASCE 41-17 Eq. 10-4.si, f′c in MPa, γ from Table 10-12 by the joint class and its hoops)

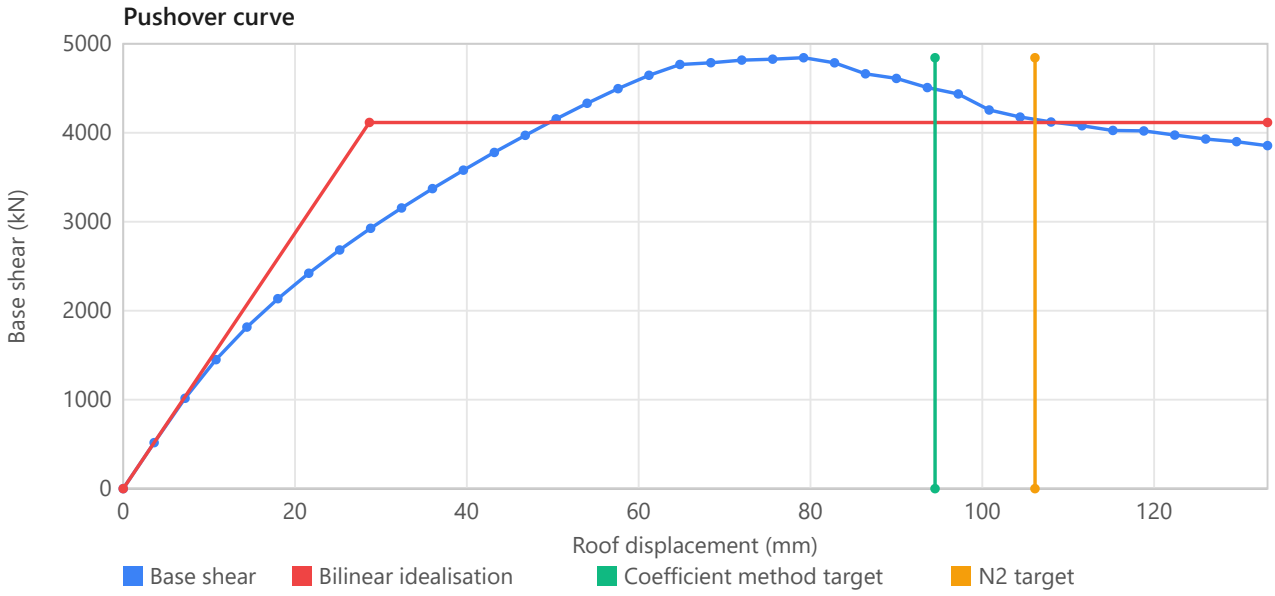
3.3.2 Pushover — Pushover Y (2400 yr)

Monotonic frame pushover with fibre sections: gravity loads first, then the triangular lateral pattern under displacement control of the roof.

3.3.2.1 Settings

Direction	global Y, control node 401
Lateral pattern	triangular: Fi ∝ wi·zi
Loading	monotonic to 3.0 % roof drift
Steps	150 (38 converged points)
Member shear check	on, the push stops at the first shear failure
Rigid joint zone	0.25 of the half-depth
Bar slip / hinge integration	no bar slip; Lobatto fibre elements (θp over L/20)

3.3.2.2 Capacity curve



Capacity curve with the bilinear idealisation and the target displacements

Point	Roof displacement (mm)	Drift	Base shear (kN)
First bar yield (bar strain ≥ fy/Es), element 3	50	0.28 %	4156
Bilinear idealisation yield point	29	0.16 %	4116
First core crushing (strain ≥ εcu), element 11	90	0.50 %	4611
Peak	79	0.44 %	4843
Ultimate (strength down to 80 % of the peak: our stop rule)	133	0.74 %	3856

The events are threshold checks on the analysis, not outcomes it reports: yield when a watched bar of a hinge-end section (the extreme bars, a sample of the section) reaches fy/Es, crushing when a watched core-corner fibre reaches εcu, shear when a member end shear reaches Vc + Vs. The push ends where the strength has fallen to 80 % of the peak (our stop rule, after the ASCE 41 / FEMA 356 ultimate point).

3.3.2.3 *Bilinear idealisation*

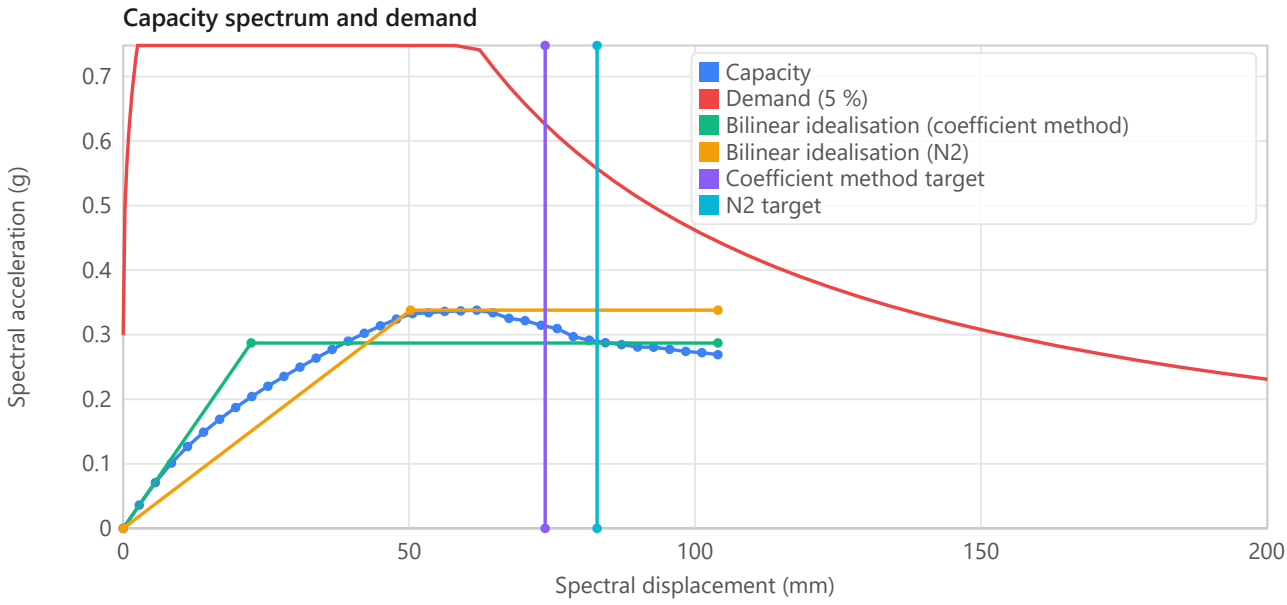
Elastic–perfectly plastic idealisation computed with the run: the initial stiffness (first step) and equal energy up to the ultimate point (the same area under both curves). The coefficient method below takes Vy from it but its own effective stiffness, the secant through the curve at 0.6 Vy; the N2 method idealises the curve its own way (Fy* the largest force).

$$
\begin{aligned}
V_y &= 4116\ \text{kN}, \Delta_y = 28.7\ \text{mm} \\
K_y &= \frac{V_y}{\Delta_y} \\
&= \frac{4116\ \text{kN}}{28.7\ \text{mm}} \\
&= 143619\ \text{kN/m} \\
\mu &= \frac{\Delta_u}{\Delta_y} \\
&= \frac{133.2}{28.7} \\
&= 4.65 \\
\frac{V_y}{W} &= \frac{4116}{16618} \\
&= 0.248 \\
\Omega &= \frac{Vm_{\text{ax}}}{V_y} \\
&= \frac{4843}{4116} \\
&= 1.18
\end{aligned}
$$

3.3.2.4 *Capacity spectrum*

The curve converted to the equivalent single-degree-of-freedom system in acceleration–displacement format with the nodal weights and the elastic pushed shape φ (roof = 1): the lateral displacements the first push step adds to the gravity state.

$$
\begin{aligned}
\Gamma &= \sum (w \cdot \varphi) \,/\, \sum (w \cdot \varphi^2) \\
&= 1.281 \\
\alpha &= \frac{(\sum w \cdot \varphi)^2}{W \cdot \sum (w \cdot \varphi^2)} \\
&= 0.862 \\
W &= 16618\ \text{kN} \\
S_d &= \frac{\Delta_{\text{roof}}}{\Gamma} \\
S_a &= \frac{V}{\alpha \cdot W} \\
T_i &= 0.560\ \text{s}
\end{aligned}
$$



Capacity spectrum against the 5 % damped design demand (ADRS)

3.3.2.5 Performance point

Method	Target roof displacement (mm)	Roof drift	Within the push
ASCE 41-17 coefficient method	94.5	0.52 %	yes
N2	106.1	0.59 %	yes

3.3.2.5.1 *Coefficient method (ASCE 41-17 7.4.3.3)*

$$K_i = \frac{V_1}{\Delta_1}$$
$$= \frac{517.0\ \text{kN}}{3.60\ \text{mm}}$$
$$= 143619\ \text{kN/m}$$
$$\Delta_e = 22.3\ \text{mm}$$

the roof displacement where the curve reaches 0.6 Vy

$$K_e = \frac{0.6 \cdot V_y}{\Delta_e}$$
$$= \frac{0.6 \cdot 4116\ \text{kN}}{22.3\ \text{mm}}$$
$$= 110913\ \text{kN/m}$$
$$T_i = 2\pi \cdot \sqrt{\frac{S_{\text{d1}}}{S_{\text{a1}} \cdot g}}$$
$$= 0.560\ \text{s}$$
$$T_e = T_i \cdot \sqrt{\frac{K_i}{K_e}}$$
$$= 0.560 \cdot \sqrt{\frac{143619}{110913}}$$
$$= 0.637\ \text{s}$$
$$S_a = \frac{S_{\text{D1}}}{T_e}$$
$$= \frac{0.431}{0.637}$$
$$= 0.677g$$
$$W = 16618\ \text{kN}$$
$$C_m = 1.00(g_{\text{iven}})$$
$$R = \frac{S_a}{\frac{V_y}{W}} \cdot C_m$$
$$= \frac{0.677}{\frac{4116}{16618}} \cdot 1.00$$
$$= 2.732$$
$$C_0 = \Gamma$$
$$= 1.281$$
$$\mu m_{\text{ax}} = \frac{\Delta_d}{\Delta_y} + |\alpha_e|\ \hat{}\ - \frac{h}{4}$$
$$= 7.21(\text{Eq}\ .7 - 32)$$

R ≤ μmax: the NSP is permitted

$$C_1 = 1 + \frac{R-1}{a \cdot T_e^2}$$
$$= 1 + \frac{2.732 - 1}{60 \cdot 0.637^2}$$
$$= 1.071 \ \text{(a = 60 for site class D)}$$
$$C_2 = 1 + \frac{\left(\frac{R-1}{T_e}\right)^2}{800}$$
$$= 1 + \frac{\left(\frac{2.732-1}{0.637}\right)^2}{800}$$
$$= 1.009$$
$$\delta_t = \frac{C_0 \cdot C_1 \cdot C_2 \cdot S_a \cdot T_e^2 \cdot g}{4\pi^2}$$
$$= \frac{1.281 \cdot 1.071 \cdot 1.009 \cdot 0.677 \cdot 0.637^2 \cdot 9.807}{4\pi^2}$$
$$= 94.5 \text{ mm}$$

g = 9.807 m/s²

$$\frac{\delta_t}{H} = \frac{94.5 \text{ mm}}{18.00 \text{ m}}$$
$$= 0.52\%$$

Te, C0, C1, C2, Cm and μmax are checked against ASCE/SEI 41-17 §7.4.3 (Eqs. 7-27 to 7-33, Table 7-4); αPΔ in μmax is approximated by −W/(H·Ke).

3.3.2.5.2 *N2 method (EN 1998-1 Annex B)*

$$\Gamma = \sum (m \cdot \varphi) \,/\, \sum (m \cdot \varphi^2)$$
$$= 1.281$$
$$F_y^* = 0.3380g$$

the largest spectral acceleration of the equivalent system

$$d_m^* = \frac{\Delta_u}{\Gamma}$$
$$= 104.0 \text{ mm}$$
$$E_m^* = 26.660g \cdot \text{mm}$$

the area under the curve to dm*

$$d_y^* = 2 \cdot \left(d_m^* - \frac{E_m^*}{F_y^*} \right)$$
$$= 2 \cdot \left(104.0 - \frac{26.660}{0.3380} \right)$$
$$= 50.25 \text{ mm}$$
$$T^* = 2\pi \cdot \sqrt{\frac{d_y^*}{F_y^* \cdot g}}$$
$$= 2\pi \cdot \sqrt{\frac{0.05025}{0.3380 \cdot 9.807}}$$
$$= 0.774 \text{ s}$$
$$S_e = 0.557g$$

the design spectrum at T*

$$S_{\text{de}} = S_e \cdot g \cdot \left(\frac{T^*}{2\pi} \right)^2$$
$$= 0.557 \cdot 9.807 \cdot \left(\frac{0.774}{2\pi} \right)^2$$
$$= 82.9 \text{ mm}$$
$$q_u = \frac{S_e}{F_y^*}$$
$$= \frac{0.557}{0.3380}$$
$$= 1.649$$
$$T_c = \frac{S_{\text{D1}}}{S_{\text{DS}}}$$
$$= 0.576 \text{ s}$$
$$d_t^* = S_{\text{de}}$$
$$= 82.9 \text{ mm}$$

T* ≥ Tc, equal displacements

$$
\begin{aligned}
d_t &= \Gamma \cdot d_t^* \\
&= 1.281 \cdot 82.9\ \text{mm} \\
&= 106.1\ \text{mm} \\
\frac{d_t}{H} &= 0.59\%
\end{aligned}
$$

With the ASCE 7 spectrum shape and Tc = SD1/SDS; checked against a published worked example.

3.3.2.6 *Hinge acceptance at the targets*

Plastic rotation θp = (φ − φy)·Lh at each yielded member end (Lh = L/20, the end Lobatto weight, or the hinge length with hinge integration; plus bar slip beyond yield) against the ASCE 41-17 Table 10-7 / 10-8 limits interpolated on the member parameters.

$$
\theta_p = (\varphi - \varphi_y) \cdot L_h
$$

Table parameters: RC beams (ρ − ρ′)/ρbal and V/(bw d √f′c) in psi units, conforming ties s ≤ d/3; RC columns P/(Ag f′c), the transverse ratio ρt = Av/(b s) (≤ 0.0175) and VyE/VColOE (≥ 0.2), IO 0.15a ≤ 0.005, LS 0.5b, CP 0.7b with b at P/(Ag f′c) ≥ 0.1 (Table 10-9 for circular columns with hoops); steel θy = Mpe L/(6 E I), columns Mpce L/(6 τb E I) at the gravity axial force.

The limits and coefficients are checked against ASCE/SEI 41-17: steel Table 9-7.1 and Eqs. 9-1 to 9-7; RC beams Table 10-7; RC columns Tables 10-8 and 10-9 with their notes and Eq. 10-3; RC walls Table 10-19; joints Table 10-12 and Eq. 10-4; target displacement Eqs. 7-27 to 7-33 and Tables 7-4 and 7-5. The FEMA 356 coefficients and N2 are checked against published worked examples. Engineering judgement: a roof joint with beams on two opposite faces is classed as exterior; the wall check assumes unconfined boundaries, low axial load and high shear because the piers' forces are not in the results.

3.3.2.6.1 *At the coefficient method target (step 27, 97 mm)*

WORST LS 330 hinges: 294 elastic, 16 IO, 20 LS, 0 CP, 0 beyond CP.

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
17	i	column	C1 storey 1	11.2	3.2	17.5	24.5	0.46	LS
11	i	column	C1 storey 1	11.2	3.2	17.5	24.5	0.46	LS
15	i	column	C1 storey 1	9.9	3.4	19.5	27.4	0.36	LS
9	i	column	C1 storey 1	9.9	3.4	19.5	27.4	0.36	LS
5	i	column	C1 storey 1	10.2	3.7	21.4	29.9	0.34	LS
23	i	column	C1 storey 1	10.2	3.7	21.4	29.9	0.34	LS
16	j	column	C1 storey 1	8.7	3.2	19.6	27.5	0.31	LS
10	j	column	C1 storey 1	8.6	3.2	19.6	27.5	0.31	LS
3	i	column	C1 storey 1	9.8	4.0	24.5	34.3	0.28	LS
21	i	column	C1 storey 1	9.8	4.0	24.5	34.3	0.28	LS
18	j	column	C1 storey 1	6.0	3.0	17.6	24.6	0.24	LS
12	j	column	C1 storey 1	6.0	3.0	17.6	24.6	0.24	LS
4	j	column	C1 storey 1	7.7	3.8	24.6	34.5	0.22	LS
22	j	column	C1 storey 1	7.7	3.8	24.6	34.5	0.22	LS
7	i	column	C1 storey 1	8.4	5.0	29.0	40.6	0.21	LS
13	i	column	C1 storey 1	8.4	5.0	29.0	40.6	0.21	LS

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
19	i	column	C1 storey 1	7.9	5.0	29.0	40.6	0.19	LS
1	i	column	C1 storey 1	7.9	5.0	29.0	40.6	0.19	LS
6	j	column	C1 storey 1	5.8	3.6	21.5	30.1	0.19	LS
24	j	column	C1 storey 1	5.8	3.5	21.5	30.1	0.19	LS
14	j	column	C1 storey 1	3.5	5.0	29.0	40.6	0.09	IO
8	j	column	C1 storey 1	3.5	5.0	29.0	40.6	0.09	IO
20	j	column	C1 storey 1	3.2	5.0	29.0	40.6	0.08	IO
2	j	column	C1 storey 1	3.2	5.0	29.0	40.6	0.08	IO
126	i	column	C1 storey 1	0.8	3.4	22.2	31.1	0.02	IO
120	i	column	C1 storey 1	0.8	3.4	22.2	31.1	0.02	IO
127	j	column	C1 storey 1	0.6	3.5	22.3	31.3	0.02	IO
121	j	column	C1 storey 1	0.6	3.5	22.3	31.3	0.02	IO
114	i	column	C1 storey 1	0.4	3.9	26.7	37.4	0.01	IO
115	j	column	C1 storey 1	0.4	4.0	26.9	37.6	0.01	IO

6 more yielded hinges are within IO or better.

Joint shear at this step: 0 of 60 joints over capacity, worst 0.40 at node 4.

3.3.2.6.2 At the N2 target (step 30, 108 mm)

WORST LS 330 hinges: 296 elastic, 10 IO, 24 LS, 0 CP, 0 beyond CP.

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
17	i	column	C1 storey 1	13.8	3.4	18.6	26.0	0.53	LS
11	i	column	C1 storey 1	13.8	3.3	18.6	26.0	0.53	LS
5	i	column	C1 storey 1	13.8	3.7	20.9	29.3	0.47	LS
23	i	column	C1 storey 1	13.8	3.7	20.9	29.3	0.47	LS
9	i	column	C1 storey 1	13.1	3.5	21.4	30.0	0.44	LS
15	i	column	C1 storey 1	13.1	3.5	21.4	30.0	0.44	LS
10	j	column	C1 storey 1	12.5	3.4	21.5	30.2	0.41	LS
16	j	column	C1 storey 1	12.5	3.4	21.6	30.2	0.41	LS
18	j	column	C1 storey 1	10.6	3.1	18.7	26.1	0.41	LS
12	j	column	C1 storey 1	10.6	3.1	18.7	26.1	0.41	LS
3	i	column	C1 storey 1	12.9	3.9	24.5	34.3	0.38	LS
21	i	column	C1 storey 1	12.8	4.0	24.5	34.3	0.37	LS
4	j	column	C1 storey 1	11.4	3.8	24.6	34.5	0.33	LS
22	j	column	C1 storey 1	11.4	3.8	24.6	34.5	0.33	LS
6	j	column	C1 storey 1	9.4	3.5	21.0	29.4	0.32	LS
24	j	column	C1 storey 1	9.4	3.4	21.0	29.4	0.32	LS
7	i	column	C1 storey 1	11.6	5.0	29.0	40.6	0.29	LS

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
13	i	column	C1 storey 1	11.6	5.0	29.0	40.6	0.29	LS
1	i	column	C1 storey 1	10.7	5.0	29.0	40.6	0.26	LS
19	i	column	C1 storey 1	10.7	5.0	29.0	40.6	0.26	LS
8	j	column	C1 storey 1	6.8	4.8	29.0	40.6	0.17	LS
14	j	column	C1 storey 1	6.8	4.8	29.0	40.6	0.17	LS
2	j	column	C1 storey 1	6.5	5.0	29.0	40.6	0.16	LS
20	j	column	C1 storey 1	6.5	5.0	29.0	40.6	0.16	LS
126	i	column	C1 storey 1	0.7	3.6	24.4	34.1	0.02	IO
120	i	column	C1 storey 1	0.7	3.6	24.4	34.1	0.02	IO
127	j	column	C1 storey 1	0.6	3.6	24.5	34.3	0.02	IO
121	j	column	C1 storey 1	0.6	3.7	24.5	34.3	0.02	IO
114	i	column	C1 storey 1	0.4	3.9	26.9	37.7	0.01	IO
106	i	beam	B2 400×700	0.4	8.8	23.8	45.3	0.01	IO

4 more yielded hinges are within IO or better.

Joint shear at this step: 0 of 60 joints over capacity, worst 0.37 at node 24.

3.3.2.7 Member checks

3.3.2.7.1 Shear

Member shear at the larger target (step 30).

NO SHEAR FAILURE No member reached its shear capacity Vc + Vs.

Element	End	Section	V max (kN)	Vn (kN)	V / Vn	Status
126	i	C1 storey 1	576	771	0.75	OK
120	i	C1 storey 1	576	771	0.75	OK
127	j	C1 storey 1	573	771	0.74	OK
121	j	C1 storey 1	573	771	0.74	OK
114	i	C1 storey 1	514	771	0.67	OK
132	i	C1 storey 1	514	771	0.67	OK
115	j	C1 storey 1	512	771	0.66	OK
133	j	C1 storey 1	512	771	0.66	OK
15	i	C1 storey 1	506	771	0.66	OK
9	i	C1 storey 1	506	771	0.66	OK
16	j	C1 storey 1	505	771	0.66	OK
10	j	C1 storey 1	505	771	0.66	OK
18	j	C1 storey 1	500	771	0.65	OK
12	j	C1 storey 1	500	771	0.65	OK
17	i	C1 storey 1	499	771	0.65	OK

V max is the resultant section shear; Vn the smaller of the two local capacities (conservative). The run itself compares each component with its own capacity (the events above).

3.3.2.7.2 Beam–column joints

Joint shear at the larger target (step 30).

ALL WITHIN CAPACITY 60 beam–column joints; worst node 24 at Vj/Vn = 0.37.

Node	Joint	γ	Vj (kN)	Vn (kN)	Vj / Vn	Status
24	exterior with transverse beams	15	872	2372	0.37	OK
6	exterior with transverse beams	15	872	2372	0.37	OK
4	interior with transverse beams	20	1134	3162	0.36	OK
22	interior with transverse beams	20	1134	3162	0.36	OK
43	exterior with transverse beams	15	724	2372	0.31	OK
52	exterior with transverse beams	15	724	2372	0.31	OK
42	interior with transverse beams	20	949	3162	0.30	OK
51	interior with transverse beams	20	948	3162	0.30	OK
71	exterior with transverse beams	15	594	2372	0.25	OK
80	exterior with transverse beams	15	594	2372	0.25	OK
12	interior with transverse beams	20	782	3162	0.25	OK
18	interior with transverse beams	20	782	3162	0.25	OK
70	interior with transverse beams	20	757	3162	0.24	OK
79	interior with transverse beams	20	757	3162	0.24	OK
46	interior with transverse beams	20	692	3162	0.22	OK

$$V_j = \sum \frac{M}{0.9 \cdot d} - V_{\text{col}}$$

$$V_n = 0.083 \cdot \gamma \cdot \sqrt{f_c'} \cdot A_j$$ (ASCE 41-17 Eq. 10-4.si, f′c in MPa, γ from Table 10-12 by the joint class and its hoops)

3.4 Nonlinear time-history analysis

3.4.1 Time history — Time history LS (1000 yr)

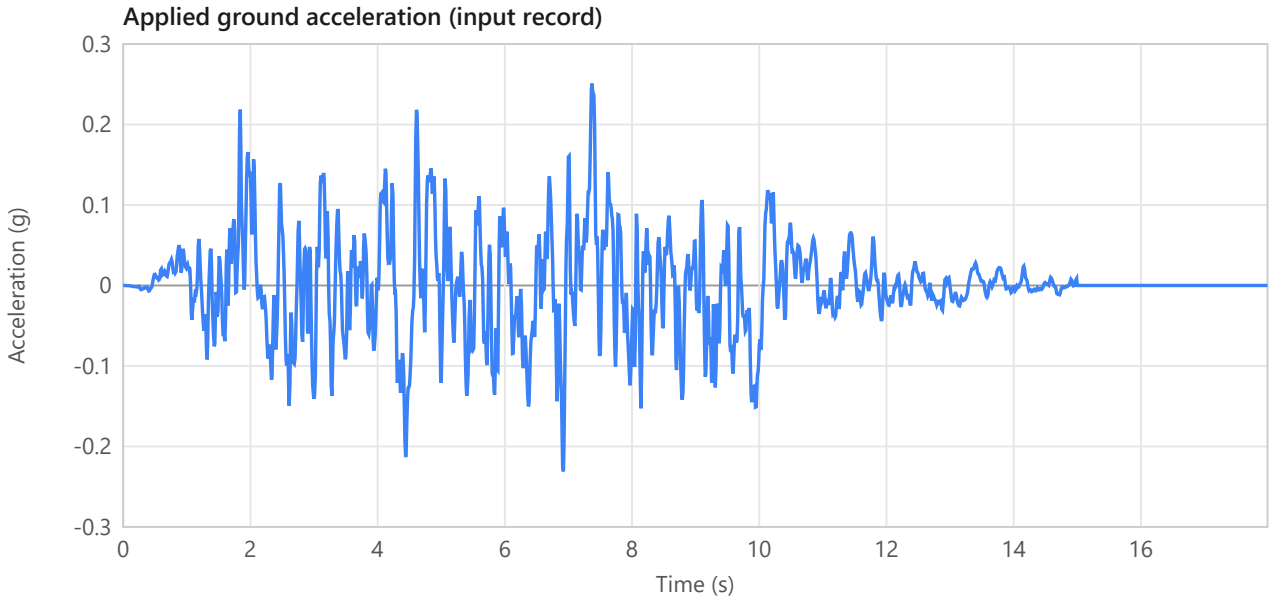
Nonlinear response history with fibre sections: gravity loads first, then record Artificial pair 1 × 1.000 in the X direction.

3.4.1.1 Ground motion

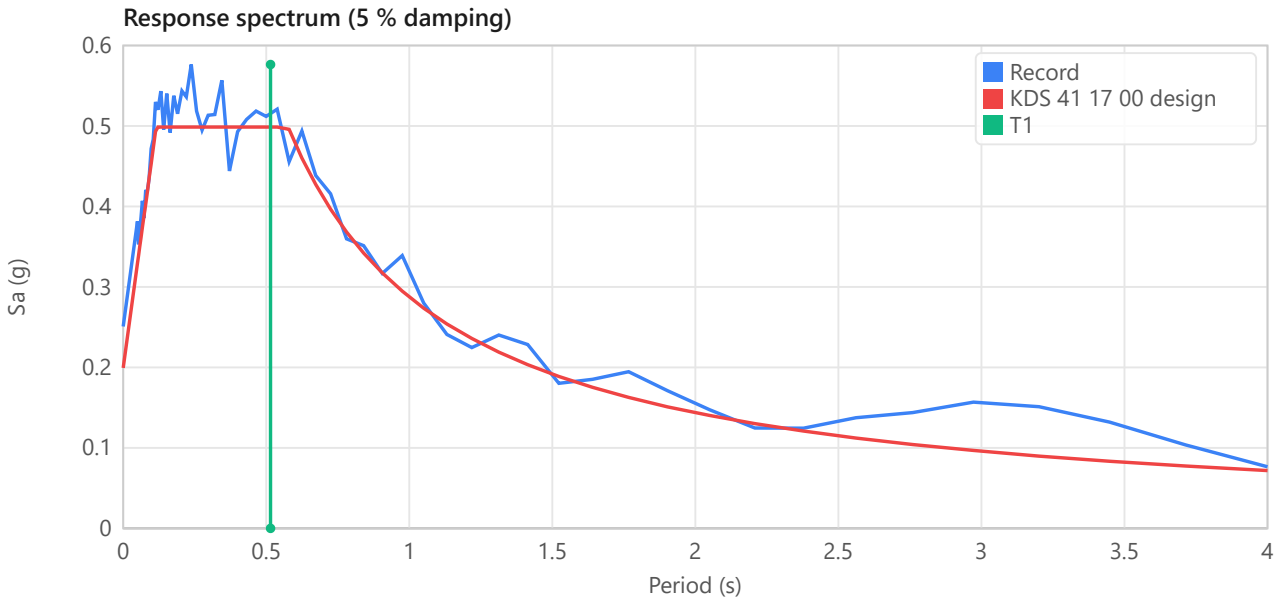
Record	Δt (s)	Points	Duration (s)	PGA (g)	Run
Artificial pair 1	0.0100	1500	15.0	0.243	× 1.000
Artificial pair 2	0.0100	1500	15.0	0.218	
Artificial pair 3	0.0100	1500	15.0	0.223	
Artificial pair 4	0.0100	1500	15.0	0.243	
Artificial pair 5	0.0100	1500	15.0	0.229	
Artificial pair 6	0.0100	1500	15.0	0.271	
Artificial pair 7	0.0100	1500	15.0	0.221	

$$P_{\mathrm{GA}} = 0.243g \cdot 1.000$$
$$= 0.251g$$
$$T_1 = 0.515\ \mathrm{s}, S_a(T_1) = 0.507g\ \text{(record)}, S_{\mathrm{aD}}(T_1) = 0.499g\ \text{(design)}$$

Scaling: the record is multiplied by 1.000 (Assign › Ground motion › Scale: one record alone to at least 90 % of the design spectrum over 0.2 T1 to 2.0 T1, or to a target Sa(T1); a code check needs a record suite). X direction, 3.0 s free-vibration tail.



Applied ground acceleration: the input record × the scale factor (not a response)



Spectrum of the scaled record against the design spectrum

3.4.1.2 Vibration modes and Rayleigh damping

Modes of the frame under its gravity loads (mass from the gravity weights, total 1694.0 t). Rayleigh damping 5.0 % at modes 2 and 5 (the dominant one and the one that reaches 90 % of the mass), committed stiffness; Newmark average acceleration (γ = 0.5, β = 0.25), steps halved down to 1/64 on non-convergence.

Mode	T (s)	f (Hz)	Effective mass	ξ
1	0.558	1.792	0.0 %	5.22 %
2	0.515	1.941	87.8 %	5.00 %
3	0.461	2.170	0.0 %	4.75 %
4	0.180	5.570	0.0 %	4.83 %
5	0.167	5.977	8.8 %	5.00 %
6	0.149	6.734	0.0 %	5.34 %
7	0.102	9.837	0.0 %	6.96 %
8	0.096	10.375	2.3 %	7.26 %
9	0.085	11.769	0.0 %	8.05 %
10	0.079	12.612	0.0 %	8.54 %
11	0.070	14.224	0.0 %	9.50 %
12	0.069	14.542	0.2 %	9.69 %

$$\omega_A = \frac{2\pi}{T_A} = \frac{2\pi}{0.515} = 12.198$$

rad/s

$$\omega_B = \frac{2\pi}{T_B} = \frac{2\pi}{0.167} = 37.552$$

rad/s

$$\alpha = \frac{2 \cdot \xi \cdot \omega_A \cdot \omega_B}{\omega_A + \omega_B} = \frac{2 \cdot 0.050 \cdot 12.198 \cdot 37.552}{12.198 + 37.552} = 0.9207$$

1/s

$$\beta = \frac{2 \cdot \xi}{\omega_A + \omega_B} = \frac{2 \cdot 0.050}{12.198 + 37.552} = 0.002010$$

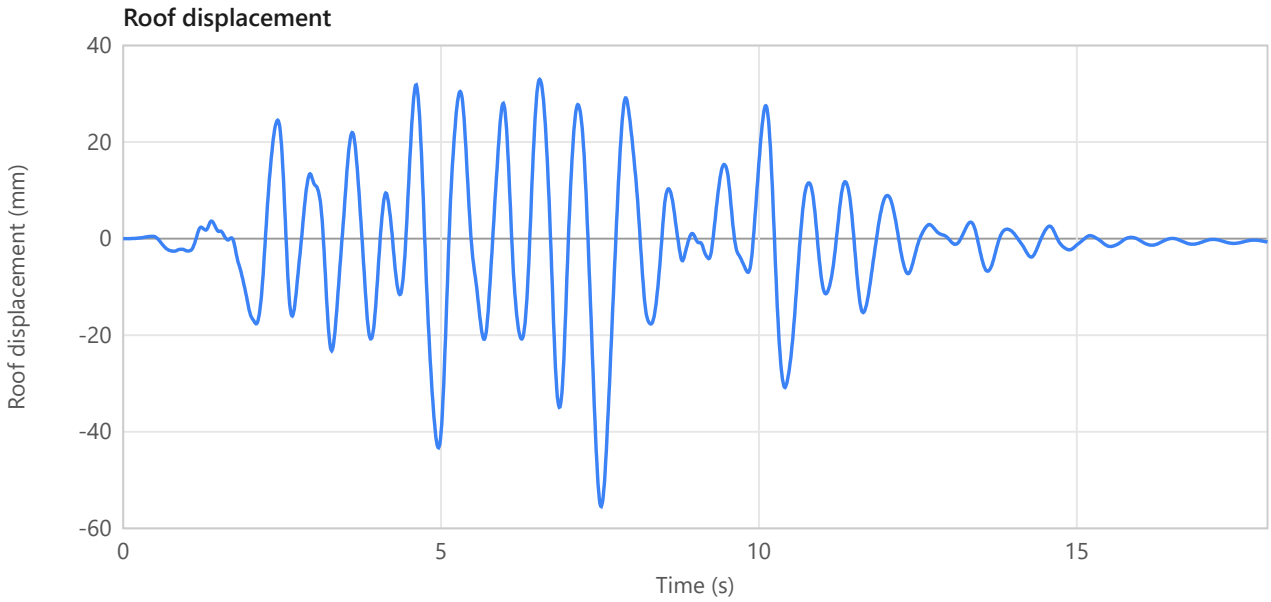
s

$$\xi_n = \frac{\alpha}{2 \cdot \omega_n} + \frac{\beta \cdot \omega_n}{2}$$

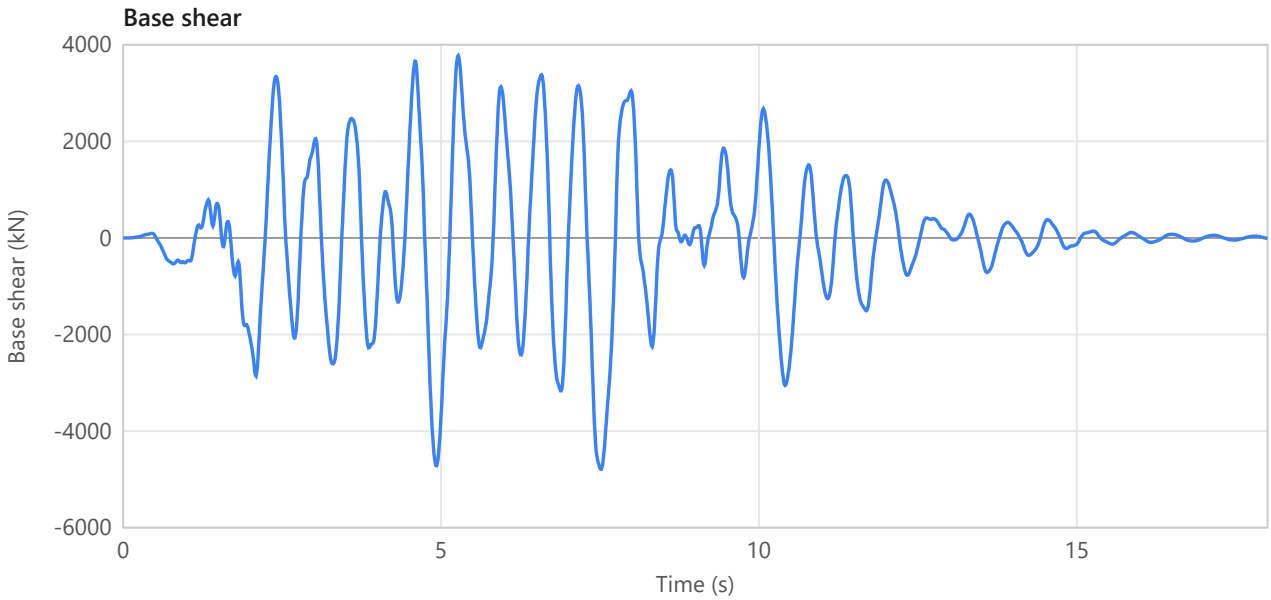
3.4.1.3 Response

NO COLLAPSE The frame survived the record.

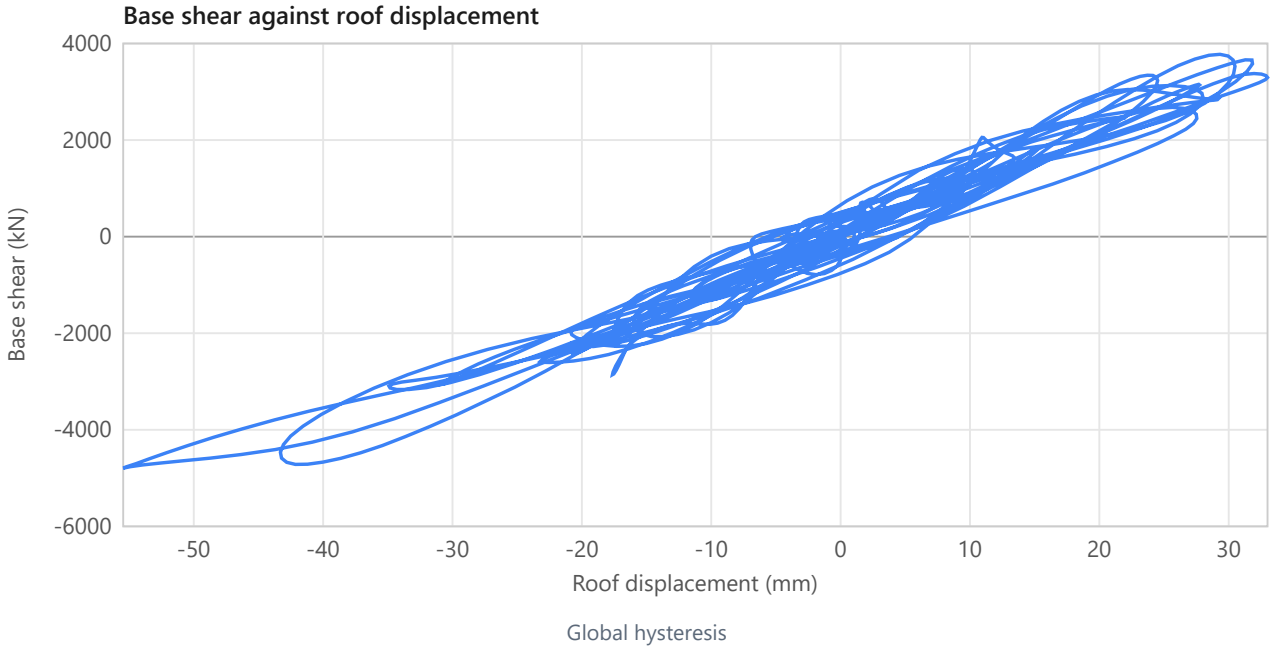
Peak roof displacement	55 mm (0.31 % drift) at 7.52 s
Peak base shear	4792 kN at 7.51 s (0.288 W)
Largest storey drift	0.60 %
Residual roof drift	0.00 %
First bar yield	element 1 at 4.88 s



Roof displacement history



Base shear history (restoring force: element resisting forces at the supports, without damping or inertia)



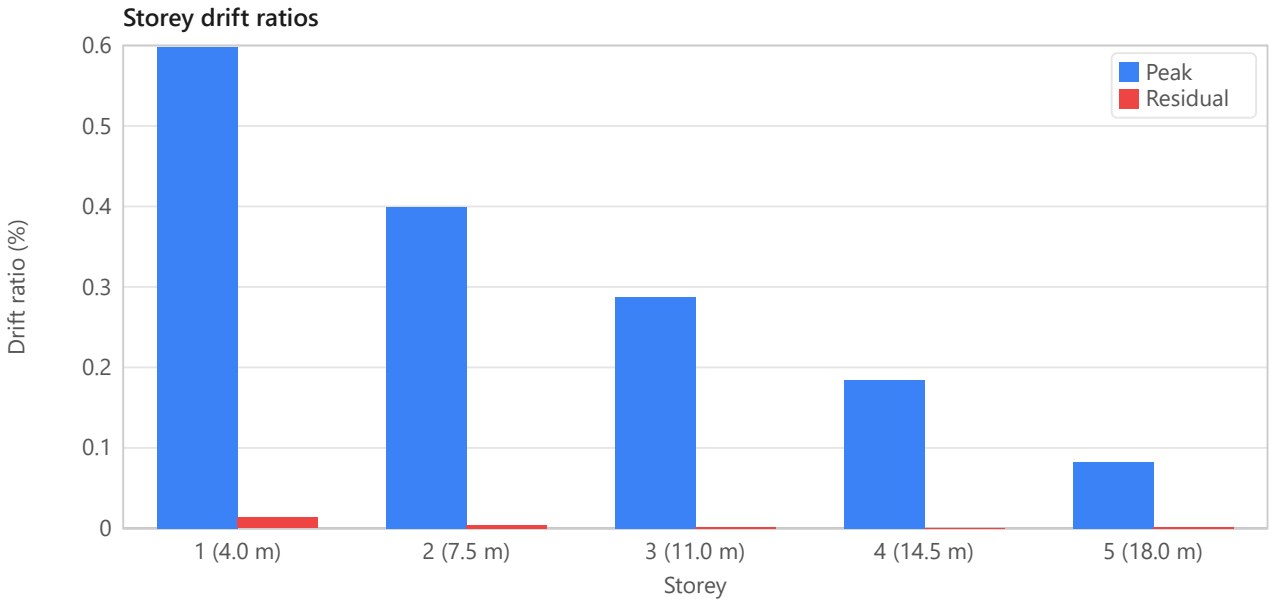
Global hysteresis

3.4.1.4 Storey drifts

Storey	Floor z (m)	Height (m)	Peak drift	At (s)	Residual drift
1	4.00	4.00	0.60 %	7.52	0.01 %
2	7.50	3.50	0.40 %	7.49	0.00 %
3	11.00	3.50	0.29 %	7.52	0.00 %
4	14.50	3.50	0.18 %	7.54	0.00 %
5	18.00	3.50	0.08 %	7.54	0.00 %

$$\theta = \frac{\Delta_i}{h_i}$$
$$= \frac{u_i - u_i - 1}{h_i}$$

$\theta m_{\text{ax}} = 0.60\,\% \leq 2.00\,\%$ (KDS 41 17 00, the objective of 1000 years)



Peak and residual storey drift ratios

3.4.1.5 *Member checks over the run*

3.4.1.5.1 *Hinge rotations*

WORST IO 330 hinges: 292 elastic, 38 IO, 0 LS, 0 CP, 0 beyond CP — the largest curvature of each hinge over the run.

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
3	i	column	C1 storey 1	2.5	3.3	18.9	26.5	0.10	IO
15	i	column	C1 storey 1	2.5	3.4	19.8	27.7	0.09	IO
9	i	column	C1 storey 1	2.6	3.5	20.4	28.6	0.09	IO
5	i	column	C1 storey 1	1.9	3.7	20.6	28.9	0.07	IO
17	i	column	C1 storey 1	1.9	3.8	21.8	30.5	0.06	IO
11	i	column	C1 storey 1	1.9	3.9	22.5	31.6	0.06	IO
7	i	column	C1 storey 1	2.3	4.1	28.9	40.4	0.06	IO
13	i	column	C1 storey 1	2.2	4.1	27.9	39.1	0.06	IO
1	i	column	C1 storey 1	2.0	4.0	25.9	36.2	0.06	IO
21	i	column	C1 storey 1	2.1	4.9	29.0	40.6	0.05	IO
19	i	column	C1 storey 1	1.9	5.0	29.0	40.6	0.05	IO
16	j	column	C1 storey 1	0.7	3.3	19.8	27.8	0.03	IO
10	j	column	C1 storey 1	0.7	3.4	20.5	28.7	0.03	IO
23	i	column	C1 storey 1	0.9	5.0	29.0	40.6	0.02	IO
18	j	column	C1 storey 1	0.4	3.6	21.9	30.7	0.01	IO
12	j	column	C1 storey 1	0.4	3.8	22.6	31.7	0.01	IO
121	j	column	C1 storey 1	0.4	3.7	22.9	32.0	0.01	IO
127	j	column	C1 storey 1	0.4	3.7	23.3	32.6	0.01	IO
14	j	column	C1 storey 1	0.4	4.1	28.1	39.3	0.01	IO

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
8	j	column	C1 storey 1	0.5	4.2	29.0	40.6	0.01	IO
4	j	column	C1 storey 1	0.3	3.2	19.0	26.5	0.01	IO
22	j	column	C1 storey 1	0.4	4.4	29.0	40.6	0.01	IO
120	i	column	C1 storey 1	0.3	3.7	22.8	31.9	0.01	IO
126	i	column	C1 storey 1	0.3	3.7	23.2	32.5	0.01	IO
115	j	column	C1 storey 1	0.2	3.8	23.2	32.4	0.01	IO
114	i	column	C1 storey 1	0.1	3.8	23.1	32.3	0.00	IO
133	j	column	C1 storey 1	0.2	4.2	29.0	40.6	0.00	IO
20	j	column	C1 storey 1	0.1	5.0	29.0	40.6	0.00	IO
129	j	column	C1 storey 1	0.1	4.0	25.3	35.5	0.00	IO
123	j	column	C1 storey 1	0.1	3.9	25.3	35.4	0.00	IO

8 more yielded hinges are within IO or better.

3.4.1.5.2 Shear

NO SHEAR FAILURE No member reached its shear capacity Vc + Vs.

Element	End	Section	V max (kN)	Vn (kN)	V / Vn	Status
126	i	C1 storey 1	543	771	0.70	OK
120	i	C1 storey 1	541	771	0.70	OK
127	j	C1 storey 1	540	771	0.70	OK
121	j	C1 storey 1	538	771	0.70	OK
3	i	C1 storey 1	489	771	0.63	OK
15	i	C1 storey 1	488	771	0.63	OK
4	j	C1 storey 1	487	771	0.63	OK
16	j	C1 storey 1	486	771	0.63	OK
132	i	C1 storey 1	483	771	0.63	OK
133	j	C1 storey 1	481	771	0.62	OK
9	i	C1 storey 1	480	771	0.62	OK
52	i	B4 400×700 T 1750×200	218	350	0.62	OK
10	j	C1 storey 1	480	771	0.62	OK
8	j	C1 storey 1	475	771	0.62	OK
14	j	C1 storey 1	475	771	0.62	OK

V max is the resultant section shear; Vn the smaller of the two local capacities (conservative). The run itself compares each component with its own capacity (the events above).

3.4.1.5.3 Beam–column joints

With the member forces at the peak roof displacement.

ALL WITHIN CAPACITY 60 beam–column joints; worst node 6 at Vj/Vn = 0.51.

Node	Joint	γ	Vj (kN)	Vn (kN)	Vj / Vn	Status
6	exterior with transverse beams	15	1209	2372	0.51	OK
4	interior with transverse beams	20	1475	3162	0.47	OK
2	exterior with transverse beams	15	1029	2372	0.43	OK
18	interior with transverse beams	20	1340	3162	0.42	OK
16	interior with transverse beams	20	1324	3162	0.42	OK
12	interior with transverse beams	20	1304	3162	0.41	OK
10	interior with transverse beams	20	1280	3162	0.40	OK
43	exterior with transverse beams	15	946	2372	0.40	OK
42	interior with transverse beams	20	1170	3162	0.37	OK
49	interior with transverse beams	20	1091	3162	0.35	OK
14	interior with transverse beams	20	1091	3162	0.34	OK
46	interior with transverse beams	20	1074	3162	0.34	OK
45	interior with transverse beams	20	1070	3162	0.34	OK
48	interior with transverse beams	20	1064	3162	0.34	OK
8	interior with transverse beams	20	1064	3162	0.34	OK

$$V_j = \sum \frac{M}{0.9 \cdot d} - V_{\text{col}}$$

$V_n = 0.083 \cdot \gamma \cdot \sqrt{f'_c} \cdot A_j$ (ASCE 41-17 Eq. 10-4.si, f′c in MPa, γ from Table 10-12 by the joint class and its hoops)

3.4.2 Time history — Time history CP (2400 yr)

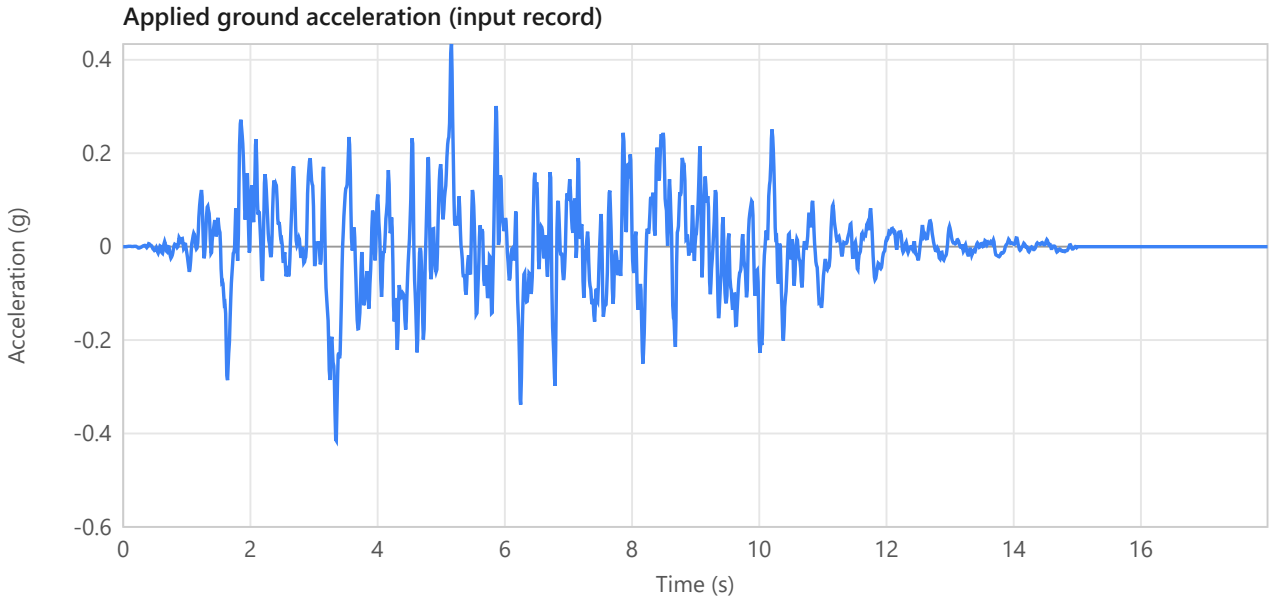
Nonlinear response history with fibre sections: gravity loads first, then record Artificial pair 1 × 1.000 in the X direction.

3.4.2.1 Ground motion

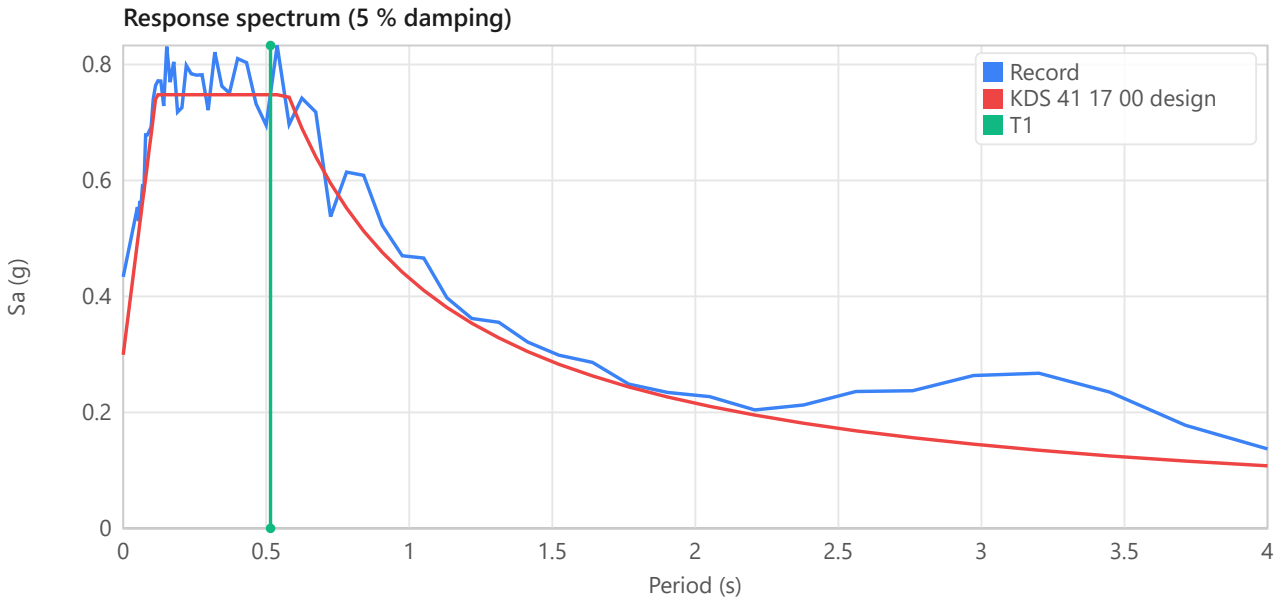
Record	Δt (s)	Points	Duration (s)	PGA (g)	Run
Artificial pair 1	0.0100	1500	15.0	0.415	× 1.000
Artificial pair 2	0.0100	1500	15.0	0.362	
Artificial pair 3	0.0100	1500	15.0	0.353	
Artificial pair 4	0.0100	1500	15.0	0.355	
Artificial pair 5	0.0100	1500	15.0	0.347	
Artificial pair 6	0.0100	1500	15.0	0.375	
Artificial pair 7	0.0100	1500	15.0	0.356	

$$P_{\mathrm{GA}} = 0.415g \cdot 1.000$$
$$= 0.434g$$
$$T_1 = 0.515\ \mathrm{s}, S_a(T_1) = 0.792g\ \text{(record)}, S_{\mathrm{aD}}(T_1) = 0.748g\ \text{(design)}$$

Scaling: the record is multiplied by 1.000 (Assign › Ground motion › Scale: one record alone to at least 90 % of the design spectrum over 0.2 T1 to 2.0 T1, or to a target Sa(T1); a code check needs a record suite). X direction, 3.0 s free-vibration tail.



Applied ground acceleration: the input record × the scale factor (not a response)



Spectrum of the scaled record against the design spectrum

3.4.2.2 Vibration modes and Rayleigh damping

Modes of the frame under its gravity loads (mass from the gravity weights, total 1694.0 t). Rayleigh damping 5.0 % at modes 2 and 5 (the dominant one and the one that reaches 90 % of the mass), committed stiffness; Newmark average acceleration (γ = 0.5, β = 0.25), steps halved down to 1/64 on non-convergence.

Mode	T (s)	f (Hz)	Effective mass	ξ
1	0.558	1.792	0.0 %	5.22 %
2	0.515	1.941	87.8 %	5.00 %
3	0.461	2.170	0.0 %	4.75 %
4	0.180	5.570	0.0 %	4.83 %
5	0.167	5.977	8.8 %	5.00 %
6	0.149	6.734	0.0 %	5.34 %
7	0.102	9.837	0.0 %	6.96 %
8	0.096	10.375	2.3 %	7.26 %
9	0.085	11.769	0.0 %	8.05 %
10	0.079	12.612	0.0 %	8.54 %
11	0.070	14.224	0.0 %	9.50 %
12	0.069	14.542	0.2 %	9.69 %

$$\omega_A = \frac{2\pi}{T_A} = \frac{2\pi}{0.515} = 12.198$$

rad/s

$$\omega_B = \frac{2\pi}{T_B} = \frac{2\pi}{0.167} = 37.552$$

rad/s

$$\alpha = \frac{2 \cdot \xi \cdot \omega_A \cdot \omega_B}{\omega_A + \omega_B} = \frac{2 \cdot 0.050 \cdot 12.198 \cdot 37.552}{12.198 + 37.552} = 0.9207$$

1/s

$$\beta = \frac{2 \cdot \xi}{\omega_A + \omega_B} = \frac{2 \cdot 0.050}{12.198 + 37.552} = 0.002010$$

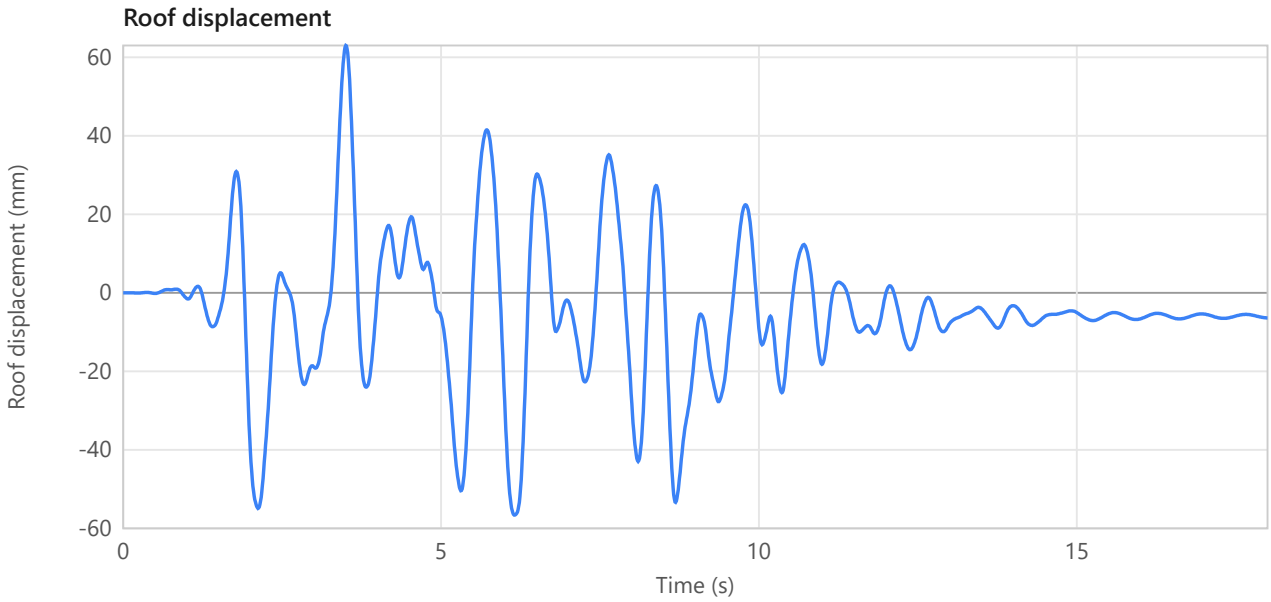
s

$$\xi_n = \frac{\alpha}{2 \cdot \omega_n} + \frac{\beta \cdot \omega_n}{2}$$

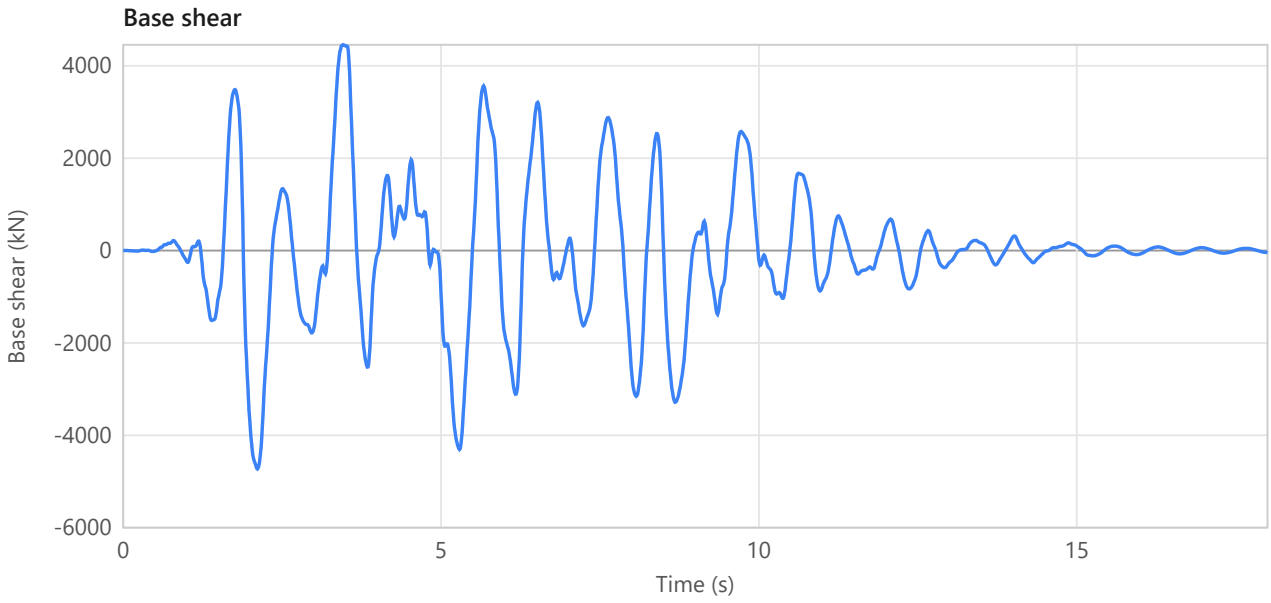
3.4.2.3 ***Response***

NO COLLAPSE The frame survived the record.

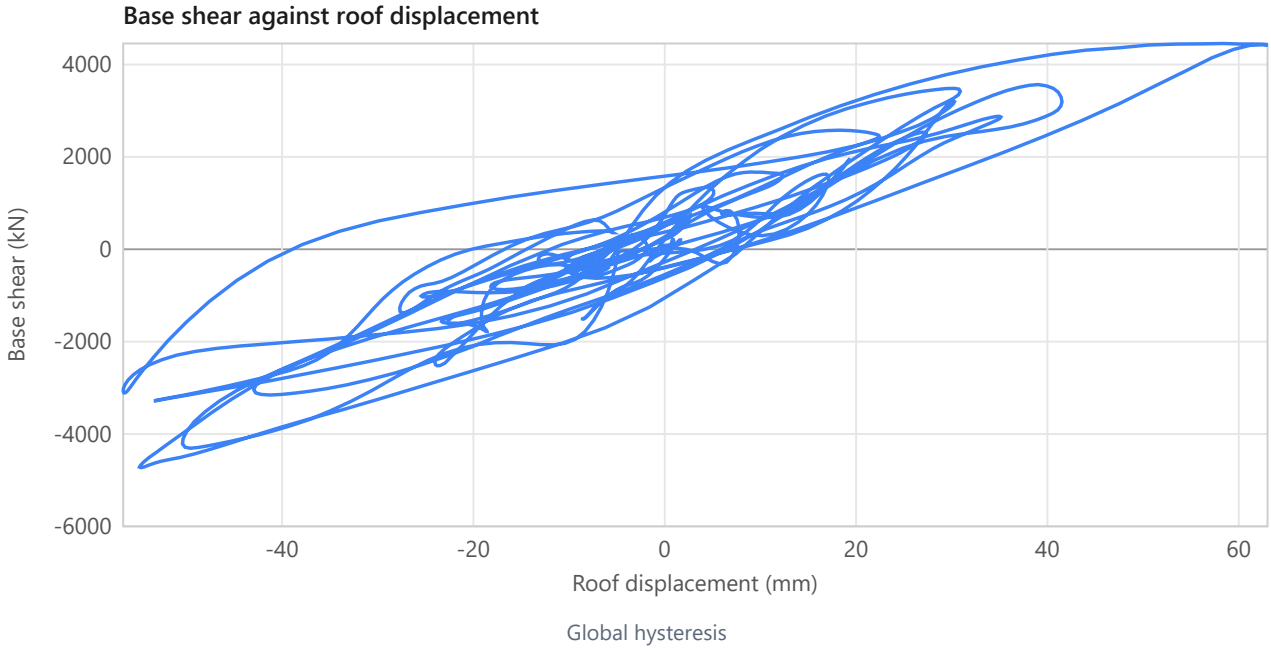
Peak roof displacement	63 mm (0.35 % drift) at 3.50 s
Peak base shear	4729 kN at 2.11 s (0.285 W)
Largest storey drift	1.19 %
Residual roof drift	0.04 %
First bar yield	element 1 at 2.00 s



Roof displacement history



Base shear history (restoring force: element resisting forces at the supports, without damping or inertia)

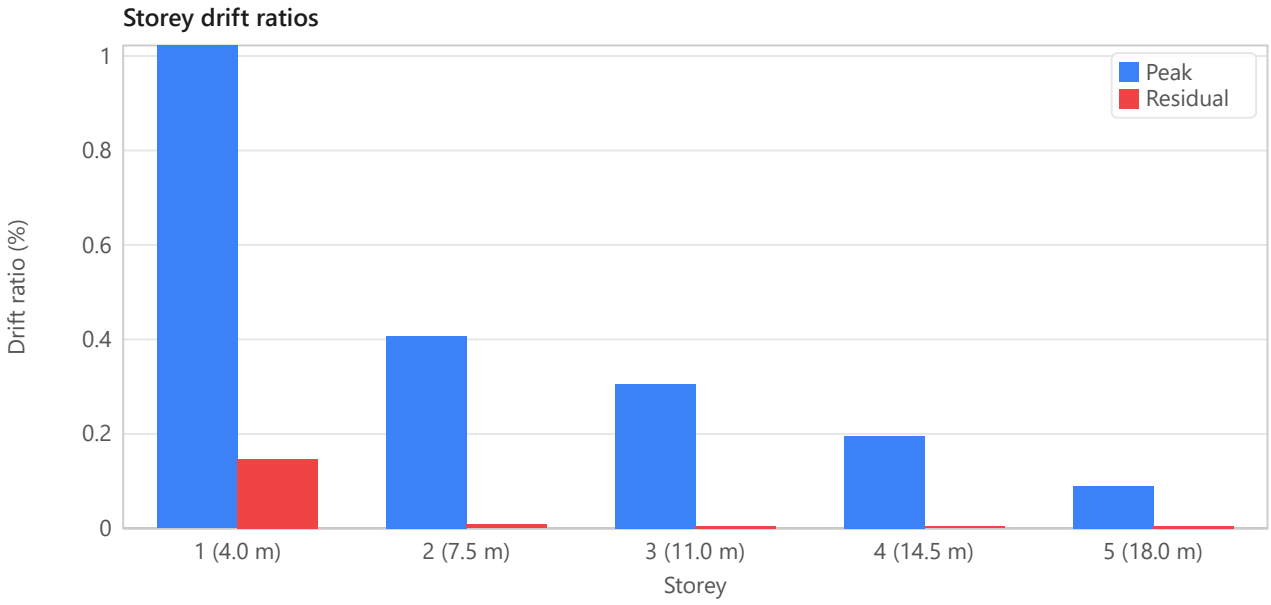


3.4.2.4 Storey drifts

Storey	Floor z (m)	Height (m)	Peak drift	At (s)	Residual drift
1	4.00	4.00	1.02 %	6.18	0.15 %
2	7.50	3.50	0.41 %	3.47	0.01 %
3	11.00	3.50	0.31 %	3.48	0.01 %
4	14.50	3.50	0.19 %	3.50	0.00 %
5	18.00	3.50	0.09 %	3.52	0.00 %

$$\theta = \frac{\Delta_i}{h_i}$$
$$= \frac{u_i - u_i - 1}{h_i}$$

$\theta m_{\text{ax}} = 1.19\,\% \leq 3.00\,\%$ (KDS 41 17 00, the objective of 2400 years)



Peak and residual storey drift ratios

3.4.2.5 *Member checks over the run*

3.4.2.5.1 *Hinge rotations*

WORST LS 330 hinges: 288 elastic, 18 IO, 24 LS, 0 CP, 0 beyond CP — the largest curvature of each hinge over the run.

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
21	i	column	C1 storey 1	12.3	3.9	18.6	26.0	0.47	LS
9	i	column	C1 storey 1	13.0	3.5	20.6	28.8	0.45	LS
15	i	column	C1 storey 1	12.9	3.5	20.9	29.2	0.44	LS
10	j	column	C1 storey 1	12.1	3.4	20.7	29.0	0.42	LS
16	j	column	C1 storey 1	11.9	3.4	21.0	29.4	0.40	LS
11	i	column	C1 storey 1	12.3	4.0	22.2	31.1	0.40	LS
23	i	column	C1 storey 1	11.1	4.4	20.1	28.2	0.39	LS
17	i	column	C1 storey 1	12.2	4.0	22.7	31.7	0.39	LS
19	i	column	C1 storey 1	12.5	4.5	24.7	34.6	0.36	LS
22	j	column	C1 storey 1	9.2	3.6	18.6	26.1	0.35	LS
7	i	column	C1 storey 1	13.2	4.1	27.8	39.0	0.34	LS
13	i	column	C1 storey 1	13.1	4.1	28.3	39.6	0.33	LS
1	i	column	C1 storey 1	13.4	4.5	29.0	40.6	0.33	LS
3	i	column	C1 storey 1	13.1	4.2	29.0	40.6	0.32	LS
5	i	column	C1 storey 1	12.5	4.6	29.0	40.6	0.31	LS
8	j	column	C1 storey 1	11.1	3.9	28.0	39.2	0.28	LS
4	j	column	C1 storey 1	11.4	4.1	29.0	40.6	0.28	LS
12	j	column	C1 storey 1	8.7	3.8	22.3	31.2	0.28	LS
14	j	column	C1 storey 1	11.0	3.9	28.5	39.8	0.28	LS

Element	End	Member	Section	θp (mrad)	IO (mrad)	LS (mrad)	CP (mrad)	θp / CP	Status
18	j	column	C1 storey 1	8.6	3.8	22.8	32.0	0.27	LS
2	j	column	C1 storey 1	10.1	4.4	29.0	40.6	0.25	LS
24	j	column	C1 storey 1	6.8	4.1	20.2	28.3	0.24	LS
20	j	column	C1 storey 1	8.2	4.1	24.8	34.8	0.24	LS
6	j	column	C1 storey 1	8.0	4.5	29.0	40.6	0.20	LS
120	i	column	C1 storey 1	0.2	3.5	24.3	34.0	0.01	IO
126	i	column	C1 storey 1	0.2	3.5	23.2	32.5	0.01	IO
115	j	column	C1 storey 1	0.1	4.3	29.0	40.6	0.00	IO
125	j	column	C1 storey 1	0.1	4.1	29.0	40.6	0.00	IO
119	j	column	C1 storey 1	0.1	4.2	29.0	40.6	0.00	IO
124	i	column	C1 storey 1	0.1	4.1	29.0	40.6	0.00	IO

12 more yielded hinges are within IO or better.

3.4.2.5.2 Shear

NO SHEAR FAILURE No member reached its shear capacity Vc + Vs.

Element	End	Section	V max (kN)	Vn (kN)	V / Vn	Status
120	i	C1 storey 1	542	771	0.70	OK
121	j	C1 storey 1	539	771	0.70	OK
126	i	C1 storey 1	539	771	0.70	OK
127	j	C1 storey 1	536	771	0.70	OK
124	i	C1 storey 1	489	771	0.63	OK
114	i	C1 storey 1	488	771	0.63	OK
15	i	C1 storey 1	488	771	0.63	OK
16	j	C1 storey 1	487	771	0.63	OK
3	i	C1 storey 1	486	771	0.63	OK
125	j	C1 storey 1	486	771	0.63	OK
4	j	C1 storey 1	486	771	0.63	OK
115	j	C1 storey 1	485	771	0.63	OK
118	i	C1 storey 1	484	771	0.63	OK
9	i	C1 storey 1	482	771	0.63	OK
119	j	C1 storey 1	481	771	0.62	OK

V max is the resultant section shear; Vn the smaller of the two local capacities (conservative). The run itself compares each component with its own capacity (the events above).

3.4.2.5.3 Beam–column joints

With the member forces at the peak roof displacement.

ALL WITHIN CAPACITY 60 beam–column joints; worst node 24 at Vj/Vn = 0.56.

Node	Joint	γ	Vj (kN)	Vn (kN)	Vj / Vn	Status
24	exterior with transverse beams	15	1320	2372	0.56	OK
22	interior with transverse beams	20	1513	3162	0.48	OK
20	exterior with transverse beams	15	1127	2372	0.48	OK
12	interior with transverse beams	20	1373	3162	0.43	OK
18	interior with transverse beams	20	1344	3162	0.42	OK
52	exterior with transverse beams	15	987	2372	0.42	OK
10	interior with transverse beams	20	1272	3162	0.40	OK
16	interior with transverse beams	20	1246	3162	0.39	OK
51	interior with transverse beams	20	1216	3162	0.38	OK
46	interior with transverse beams	20	1140	3162	0.36	OK
8	interior with transverse beams	20	1139	3162	0.36	OK
49	interior with transverse beams	20	1130	3162	0.36	OK
14	interior with transverse beams	20	1118	3162	0.35	OK
50	exterior with transverse beams	15	832	2372	0.35	OK
48	interior with transverse beams	20	1102	3162	0.35	OK

$$V_j = \sum \frac{M}{0.9 \cdot d} - V_{\text{col}}$$

$V_n = 0.083 \cdot \gamma \cdot \sqrt{f'_c} \cdot A_j$ (ASCE 41-17 Eq. 10-4.si, f′c in MPa, γ from Table 10-12 by the joint class and its hoops)

3.4.3 Time history — Time history OP (50 yr)

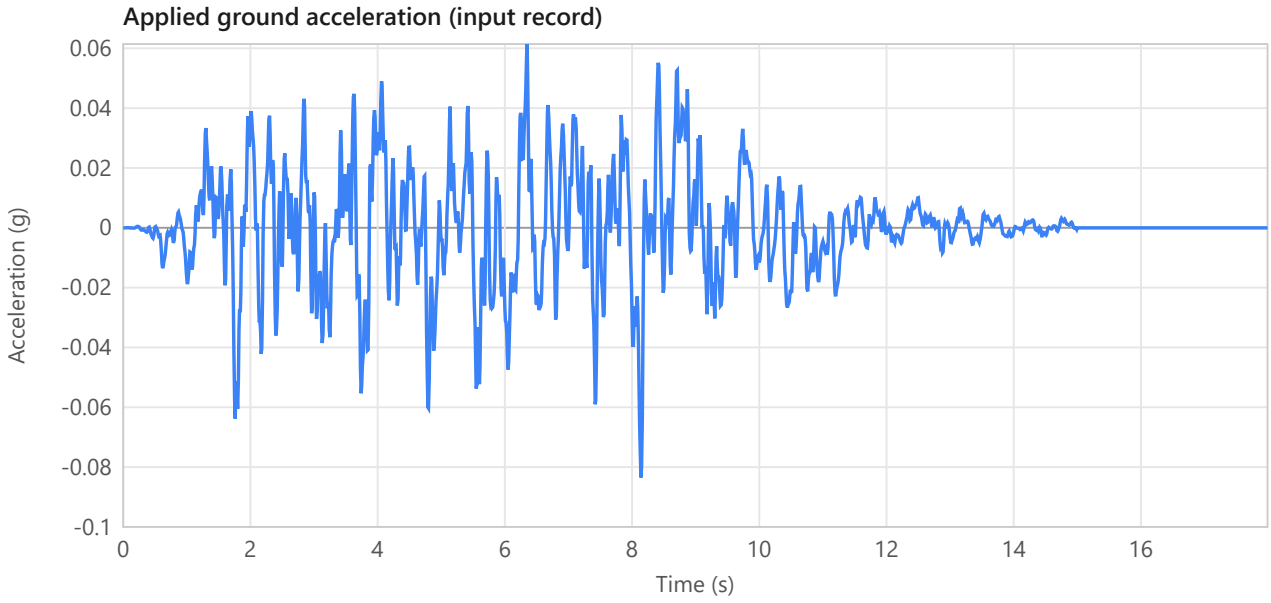
Nonlinear response history with fibre sections: gravity loads first, then record Artificial pair 1 × 1.000 in the X direction.

3.4.3.1 Ground motion

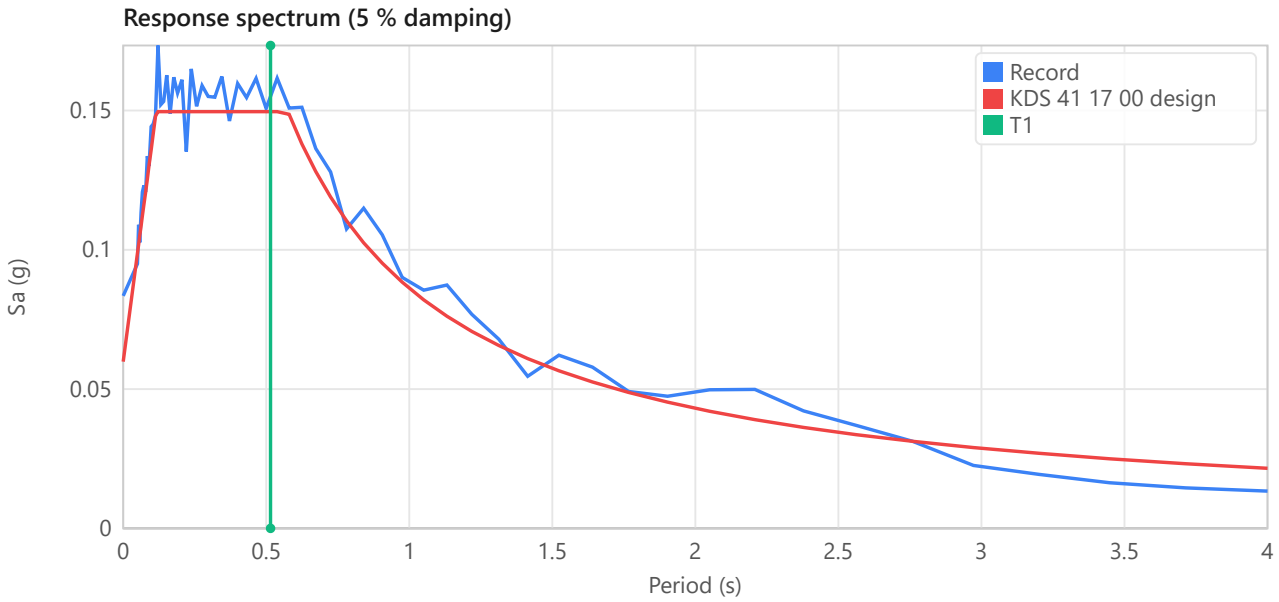
Record	Δt (s)	Points	Duration (s)	PGA (g)	Run
Artificial pair 1	0.0100	1500	15.0	0.081	× 1.000
Artificial pair 2	0.0100	1500	15.0	0.084	
Artificial pair 3	0.0100	1500	15.0	0.079	
Artificial pair 4	0.0100	1500	15.0	0.061	
Artificial pair 5	0.0100	1500	15.0	0.060	
Artificial pair 6	0.0100	1500	15.0	0.082	
Artificial pair 7	0.0100	1500	15.0	0.064	

$$P_{\mathrm{GA}} = 0.081g \cdot 1.000$$
$$= 0.083g$$
$$T_1 = 0.515\ \mathrm{s}, S_a(T_1) = 0.163g\ (\mathrm{record}), S_{\mathrm{aD}}(T_1) = 0.150g\ (\mathrm{design})$$

Scaling: the record is multiplied by 1.000 (Assign › Ground motion › Scale: one record alone to at least 90 % of the design spectrum over 0.2 T1 to 2.0 T1, or to a target Sa(T1); a code check needs a record suite). X direction, 3.0 s free-vibration tail.



Applied ground acceleration: the input record × the scale factor (not a response)



Spectrum of the scaled record against the design spectrum

3.4.3.2 Vibration modes and Rayleigh damping

Modes of the frame under its gravity loads (mass from the gravity weights, total 1694.0 t). Rayleigh damping 5.0 % at modes 2 and 5 (the dominant one and the one that reaches 90 % of the mass), committed stiffness; Newmark average acceleration (γ = 0.5, β = 0.25), steps halved down to 1/64 on non-convergence.

Mode	T (s)	f (Hz)	Effective mass	ξ
1	0.558	1.792	0.0 %	5.22 %
2	0.515	1.941	87.8 %	5.00 %
3	0.461	2.170	0.0 %	4.75 %
4	0.180	5.570	0.0 %	4.83 %
5	0.167	5.977	8.8 %	5.00 %
6	0.149	6.734	0.0 %	5.34 %
7	0.102	9.837	0.0 %	6.96 %
8	0.096	10.375	2.3 %	7.26 %
9	0.085	11.769	0.0 %	8.05 %
10	0.079	12.612	0.0 %	8.54 %
11	0.070	14.224	0.0 %	9.50 %
12	0.069	14.542	0.2 %	9.69 %

$$\omega_A = \frac{2\pi}{T_A} = \frac{2\pi}{0.515} = 12.198$$

rad/s

$$\omega_B = \frac{2\pi}{T_B} = \frac{2\pi}{0.167} = 37.552$$

rad/s

$$\alpha = \frac{2 \cdot \xi \cdot \omega_A \cdot \omega_B}{\omega_A + \omega_B} = \frac{2 \cdot 0.050 \cdot 12.198 \cdot 37.552}{12.198 + 37.552} = 0.9207$$

1/s

$$\beta = \frac{2 \cdot \xi}{\omega_A + \omega_B} = \frac{2 \cdot 0.050}{12.198 + 37.552} = 0.002010$$

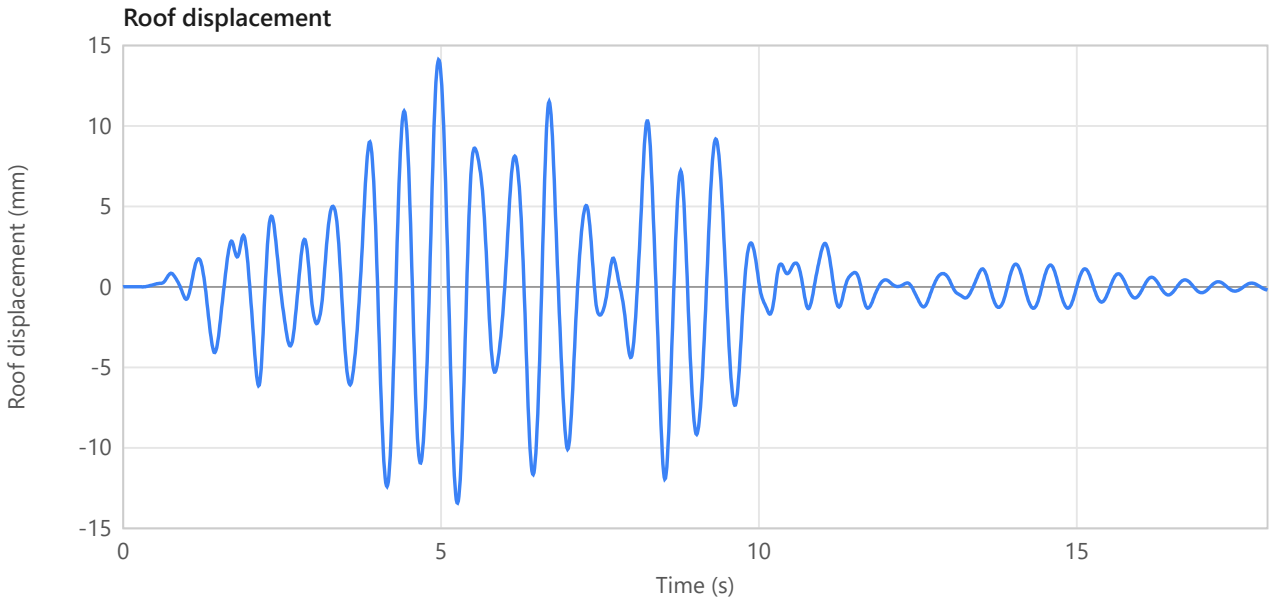
s

$$\xi_n = \frac{\alpha}{2 \cdot \omega_n} + \frac{\beta \cdot \omega_n}{2}$$

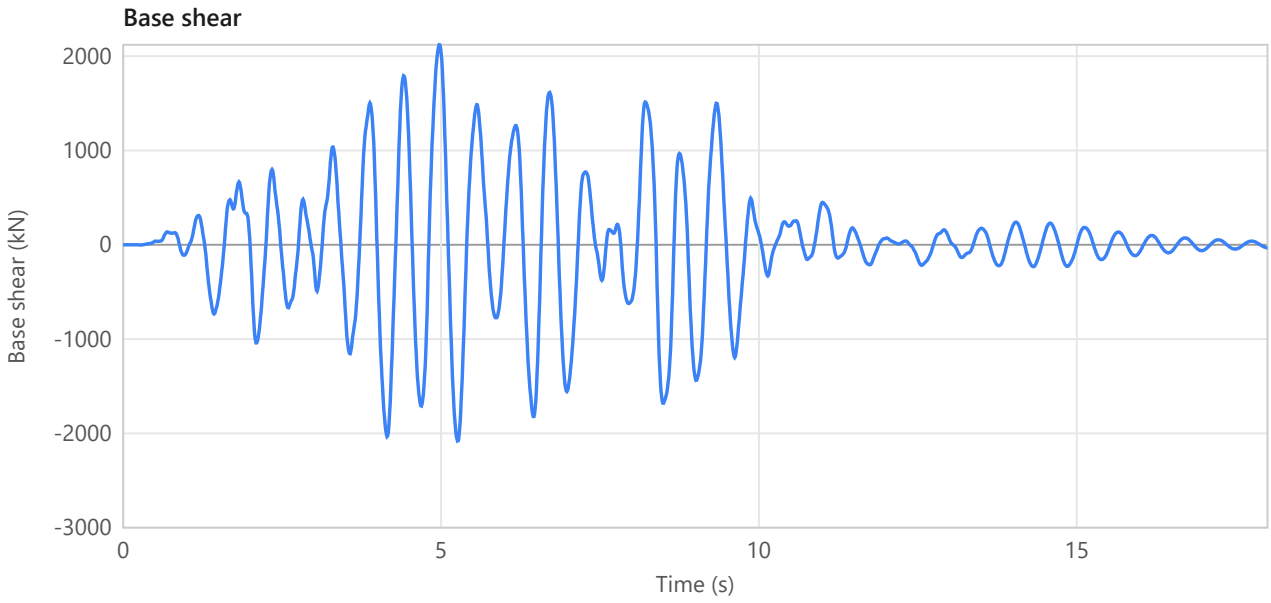
3.4.3.3 *Response*

NO COLLAPSE The frame survived the record.

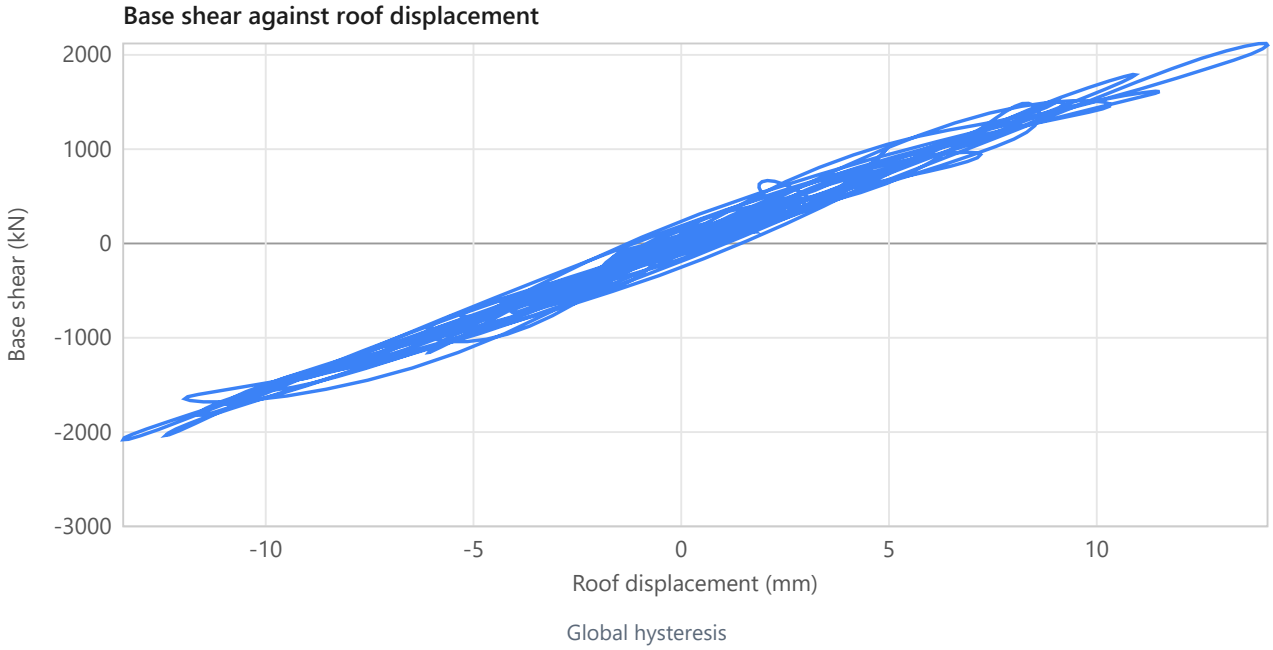
Peak roof displacement	14 mm (0.08 % drift) at 4.96 s
Peak base shear	2121 kN at 4.97 s (0.128 W)
Largest storey drift	0.12 %
Residual roof drift	0.00 %
First bar yield	none



Roof displacement history



Base shear history (restoring force: element resisting forces at the supports, without damping or inertia)

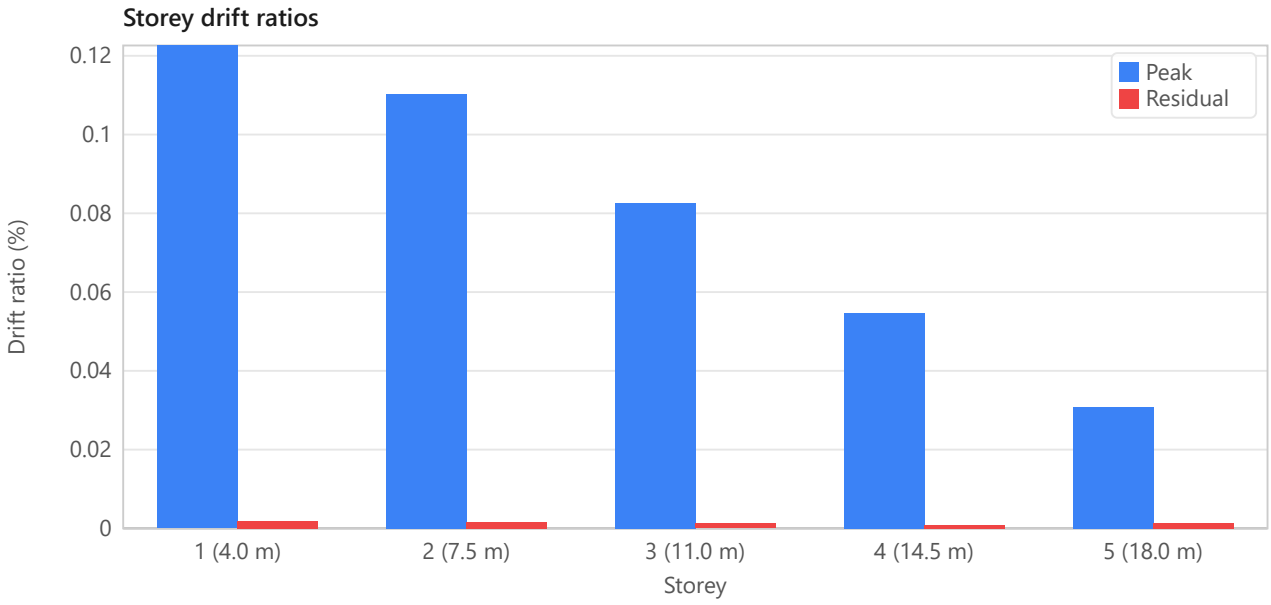


3.4.3.4 Storey drifts

Storey	Floor z (m)	Height (m)	Peak drift	At (s)	Residual drift
1	4.00	4.00	0.12 %	4.97	0.00 %
2	7.50	3.50	0.11 %	4.97	0.00 %
3	11.00	3.50	0.08 %	4.96	0.00 %
4	14.50	3.50	0.05 %	8.53	0.00 %
5	18.00	3.50	0.03 %	8.53	0.00 %

$$\theta = \frac{\Delta_i}{h_i}$$
$$= \frac{u_i - u_i - 1}{h_i}$$

$\theta m_{\text{ax}} = 0.12\,\% \leq 0.50\,\%$ (KDS 41 17 00, the objective of 50 years)



Peak and residual storey drift ratios

3.4.3.5 *Member checks over the run*

3.4.3.5.1 *Hinge rotations*

WORST ELASTIC 330 hinges: 330 elastic, 0 IO, 0 LS, 0 CP, 0 beyond CP — the largest curvature of each hinge over the run.

No member end has yielded.

3.4.3.5.2 *Shear*

NO SHEAR FAILURE No member reached its shear capacity Vc + Vs.

Element	End	Section	V max (kN)	Vn (kN)	V / Vn	Status
126	i	C1 storey 1	290	771	0.38	OK
127	j	C1 storey 1	289	771	0.37	OK
120	i	C1 storey 1	286	771	0.37	OK
60	j	B4 400×700 T 1750×200	130	350	0.37	OK
121	j	C1 storey 1	285	771	0.37	OK
52	i	B4 400×700 T 1750×200	126	350	0.36	OK
9	i	C1 storey 1	273	771	0.35	OK
10	j	C1 storey 1	273	771	0.35	OK
15	i	C1 storey 1	273	771	0.35	OK
16	j	C1 storey 1	273	771	0.35	OK
171	j	B4 400×700 T 1750×200	116	350	0.33	OK
11	i	C1 storey 1	251	771	0.33	OK
12	j	C1 storey 1	250	771	0.32	OK
21	i	C1 storey 1	250	771	0.32	OK
163	i	B4 400×700 T 1750×200	113	350	0.32	OK

V max is the resultant section shear; Vn the smaller of the two local capacities (conservative). The run itself compares each component with its own capacity (the events above).

3.4.3.5.3 Beam–column joints

With the member forces at the peak roof displacement.

ALL WITHIN CAPACITY 60 beam–column joints; worst node 24 at Vj/Vn = 0.14.

Node	Joint	γ	Vj (kN)	Vn (kN)	Vj / Vn	Status
24	exterior with transverse beams	15	340	2372	0.14	OK
16	interior with transverse beams	20	445	3162	0.14	OK
10	interior with transverse beams	20	442	3162	0.14	OK
52	exterior with transverse beams	15	327	2372	0.14	OK
18	interior with transverse beams	20	403	3162	0.13	OK
48	interior with transverse beams	20	402	3162	0.13	OK
12	interior with transverse beams	20	400	3162	0.13	OK
22	interior with transverse beams	20	395	3162	0.12	OK
49	interior with transverse beams	20	388	3162	0.12	OK
80	exterior with transverse beams	15	290	2372	0.12	OK
45	interior with transverse beams	20	371	3162	0.12	OK
46	interior with transverse beams	20	369	3162	0.12	OK
76	interior with transverse beams	20	363	3162	0.11	OK
51	interior with transverse beams	20	353	3162	0.11	OK
77	interior with transverse beams	20	344	3162	0.11	OK

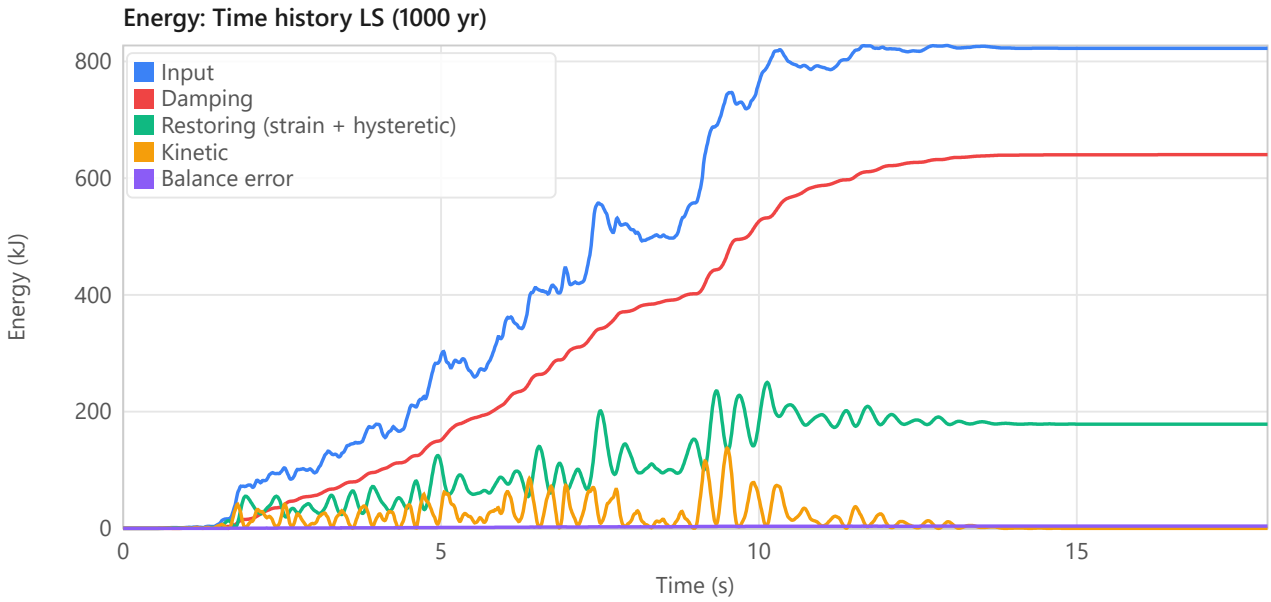
$$V_j = \sum \frac{M}{0.9 \cdot d} - V_{\text{col}}$$

$$V_n = 0.083 \cdot \gamma \cdot \sqrt{f_c'} \cdot A_j$$ (ASCE 41-17 Eq. 10-4.si, f′c in MPa, γ from Table 10-12 by the joint class and its hoops)

3.4.4 Energy balance

The energy the ground motion puts into the building (input) is held as kinetic energy and strain energy and dissipated by the viscous damping and by yielding (hysteretic). Each term is computed on its own, so their sum checks the run: input − kinetic − damping − restoring should stay near zero. At the end of the record the restoring work is the hysteretic energy, the measure of damage: a large share means the building took the earthquake by yielding rather than by damping. For a record suite the energy is that of its main record (the run kept in full).

Study	Input at the end (kJ)	Damping	Hysteretic (restoring)	Kinetic	Largest balance error
Time history LS (1000 yr)	822.5	77.8 %	21.7 %	0.0 %	0.46 %
Time history CP (2400 yr)	1898.4	44.0 %	55.6 %	0.0 %	0.33 %
Time history OP (50 yr)	101.4	91.0 %	8.4 %	0.0 %	0.59 %



Cumulative energy terms of the time history (relative frame); the balance error is input − kinetic − damping − restoring

3.5 Ground motion records and scaling

KDS 41 17 00 §7.3.4.1: at least 3 records matching the site (design on the largest response), or 7 or more (design on the mean); in 3D each pair applied twice with its components swapped, one factor for both components; the mean of the SRSS spectra not below 1.3 × the design spectrum over 0.2–1.5 T times 90 % (recorded, amplitude scaled), 110 % (spectrum matched) or 80 % (site response); in 2D 1.0 × the spectrum.

Record kind	Floor on the mean SRSS (3D)	2D
recorded motions, amplitude scaled (90 %)	1.17 ×	0.90 ×
recorded motions, spectrum matched (110 %)	1.43 ×	1.10 ×
site response analysis (80 %)	1.04 ×	0.80 ×

Time history LS (1000 yr)

#	Record	Components	Δt (s)	Duration (s)	PGA (g)	Scale factor
1	Artificial pair 1	pair	0.0100	15.0	0.243	1.030
2	Artificial pair 2	pair	0.0100	15.0	0.218	1.024
3	Artificial pair 3	pair	0.0100	15.0	0.223	1.029
4	Artificial pair 4	pair	0.0100	15.0	0.243	1.036
5	Artificial pair 5	pair	0.0100	15.0	0.229	1.021
6	Artificial pair 6	pair	0.0100	15.0	0.271	1.028
7	Artificial pair 7	pair	0.0100	15.0	0.221	1.031

Time history CP (2400 yr)

#	Record	Components	Δt (s)	Duration (s)	PGA (g)	Scale factor
1	Artificial pair 1	pair	0.0100	15.0	0.415	1.045
2	Artificial pair 2	pair	0.0100	15.0	0.362	1.040

#	Record	Components	Δt (s)	Duration (s)	PGA (g)	Scale factor
3	Artificial pair 3	pair	0.0100	15.0	0.353	1.039
4	Artificial pair 4	pair	0.0100	15.0	0.355	1.044
5	Artificial pair 5	pair	0.0100	15.0	0.347	1.030
6	Artificial pair 6	pair	0.0100	15.0	0.375	1.035
7	Artificial pair 7	pair	0.0100	15.0	0.356	1.040

Time history OP (50 yr)

#	Record	Components	Δt (s)	Duration (s)	PGA (g)	Scale factor
1	Artificial pair 1	pair	0.0100	15.0	0.081	1.032
2	Artificial pair 2	pair	0.0100	15.0	0.084	1.032
3	Artificial pair 3	pair	0.0100	15.0	0.079	1.030
4	Artificial pair 4	pair	0.0100	15.0	0.061	1.027
5	Artificial pair 5	pair	0.0100	15.0	0.060	1.040
6	Artificial pair 6	pair	0.0100	15.0	0.082	1.035
7	Artificial pair 7	pair	0.0100	15.0	0.064	1.035

4 Seismic performance evaluation

The responses against the objectives: drifts, storey response, member deformations and forces, the level reached and where the damage is.

4.1 Interstory drift ratios

The peak storey drift ratio Δ/hsx of every storey, the larger of the two directions, against the limit of the objective each study checks.

Pushover X (2400 yr)

Pushover: roof drift 0.54 % at the target roof displacement 97 mm (step 72); storey drifts come from the time histories.

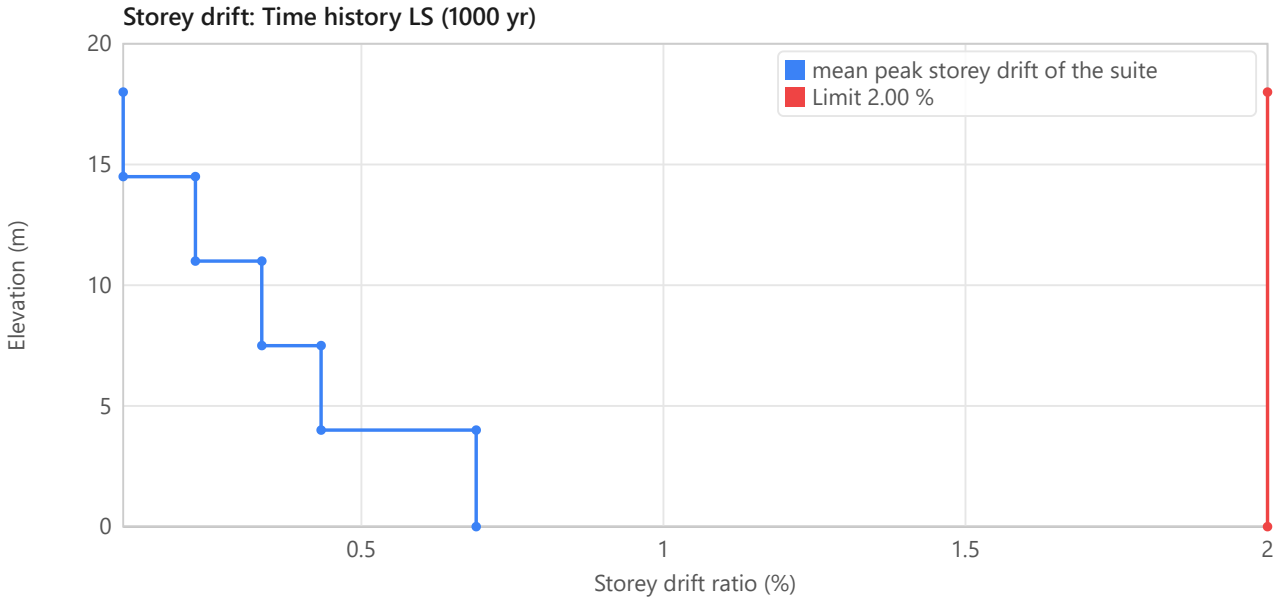
Pushover Y (2400 yr)

Pushover: roof drift 0.60 % at the target roof displacement 106 mm (step 30); storey drifts come from the time histories.

Time history LS (1000 yr)

Storey	Top z (m)	Drift	Limit	Drift / limit	Residual (shown run)
1	4.00	0.69 %	2.00 %	0.35	0.01 %
2	7.50	0.43 %	2.00 %	0.22	0.00 %
3	11.00	0.34 %	2.00 %	0.17	0.00 %
4	14.50	0.23 %	2.00 %	0.11	0.00 %
5	18.00	0.11 %	2.00 %	0.05	0.00 %

Residual drift after the free-vibration tail: largest storey value 0.01 %; largest residual roof drift of the suite's runs 0.06 %. KDS 41 17 00 sets no residual drift limit; it is reported for the repairability judgement.

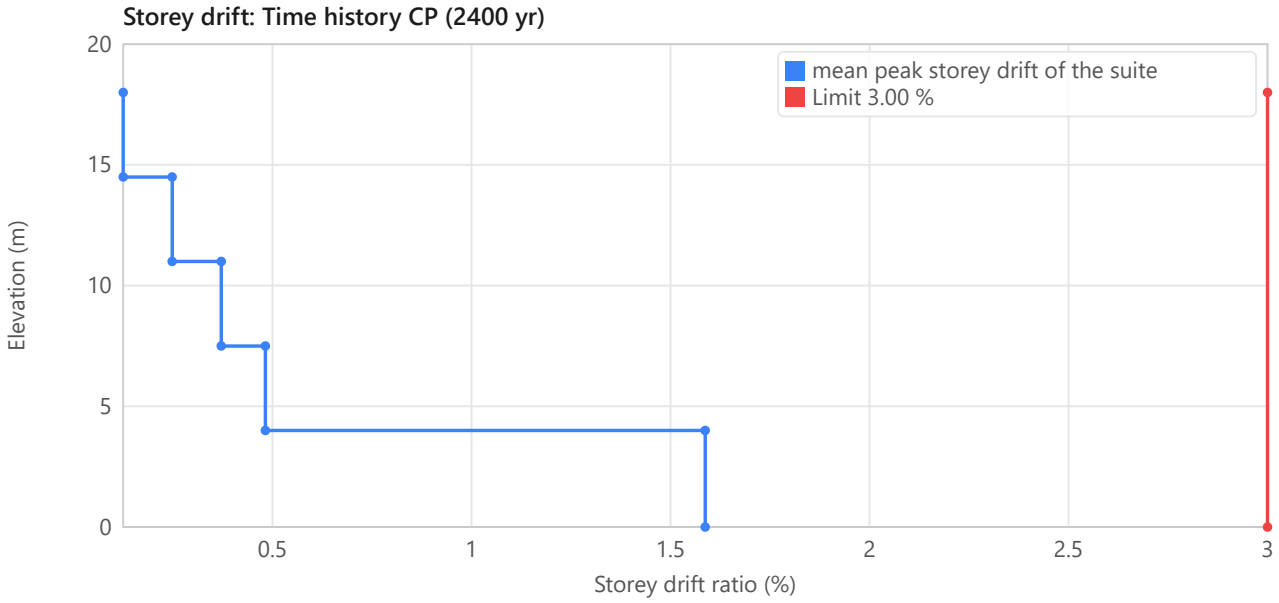


mean peak storey drift of the suite against the limit (Table 8.2-1 (§7.3.4.3, §8.2.3))

Time history CP (2400 yr)

Storey	Top z (m)	Drift	Limit	Drift / limit	Residual (shown run)
1	4.00	1.59 %	3.00 %	0.53	0.35 %
2	7.50	0.48 %	3.00 %	0.16	0.03 %
3	11.00	0.37 %	3.00 %	0.12	0.02 %
4	14.50	0.25 %	3.00 %	0.08	0.01 %
5	18.00	0.12 %	3.00 %	0.04	0.01 %

Residual drift after the free-vibration tail: largest storey value 0.35 %; largest residual roof drift of the suite's runs 0.26 %. KDS 41 17 00 sets no residual drift limit; it is reported for the repairability judgement.

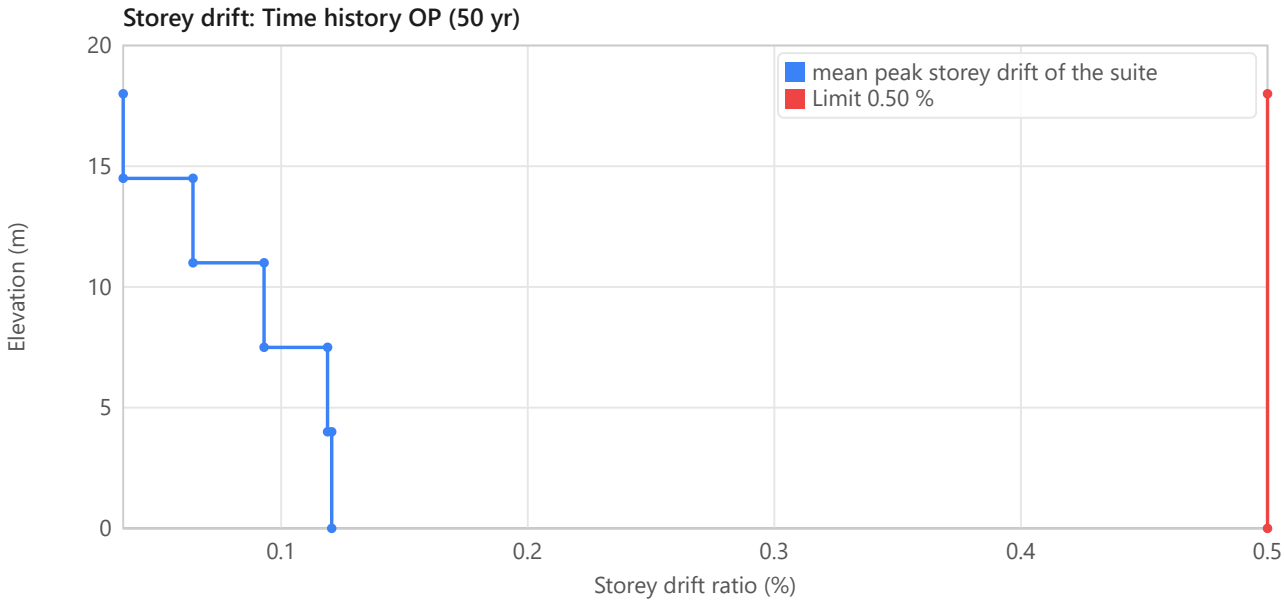


mean peak storey drift of the suite against the limit (§15.4(3): 3 % / IE)

Time history OP (50 yr)

Storey	Top z (m)	Drift	Limit	Drift / limit	Residual (shown run)
1	4.00	0.12 %	0.50 %	0.24	0.00 %
2	7.50	0.12 %	0.50 %	0.24	0.00 %
3	11.00	0.09 %	0.50 %	0.19	0.00 %
4	14.50	0.06 %	0.50 %	0.13	0.00 %
5	18.00	0.04 %	0.50 %	0.07	0.00 %

Residual drift after the free-vibration tail: largest storey value 0.00 %; largest residual roof drift of the suite's runs 0.00 %. KDS 41 17 00 sets no residual drift limit; it is reported for the repairability judgement.

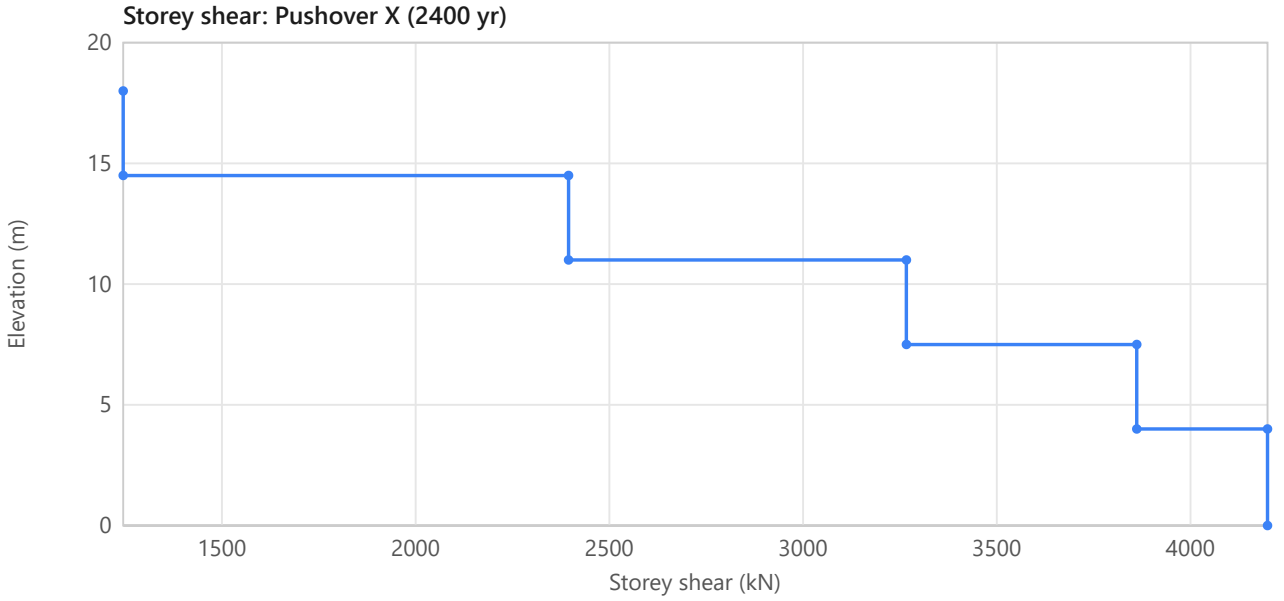


mean peak storey drift of the suite against the limit (§15.4(2))

4.2 Story-level response evaluation

Pushover X (2400 yr) (pushover, at the target roof displacement 97 mm (step 72))

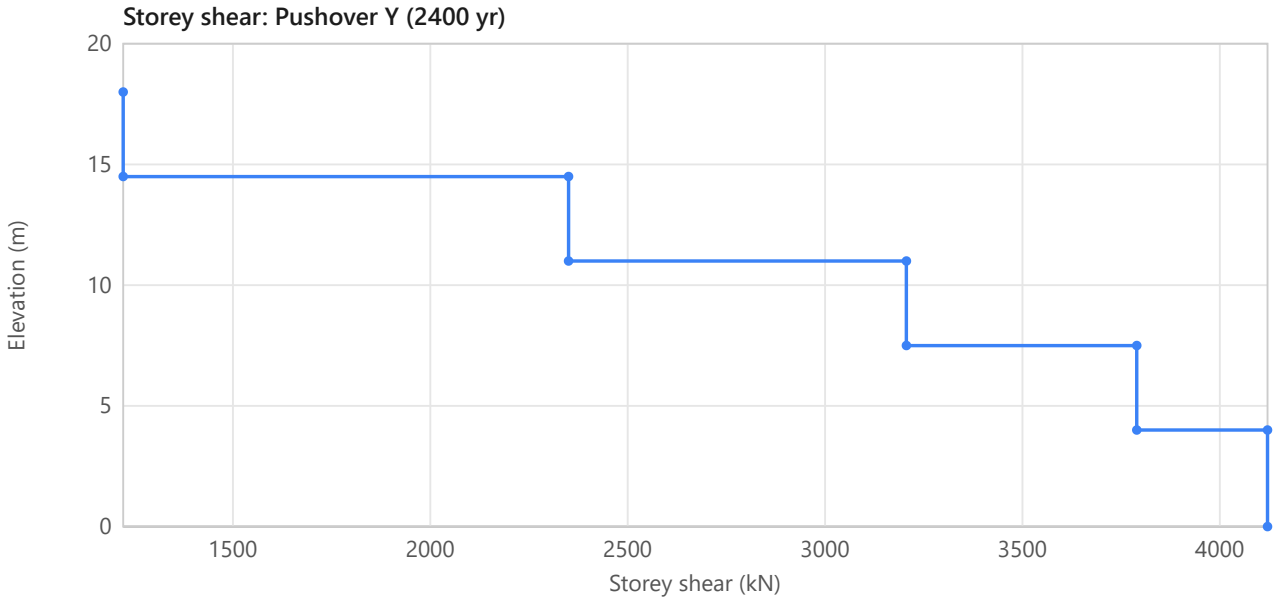
Storey	Top z (m)	Weight (kN)	Storey force (kN)	Storey shear (kN)	Overturning at its base (kN·m)
1	4.00	3582	337	4199	54482
2	7.50	3366	595	3861	37687
3	11.00	3366	872	3267	24173
4	14.50	3366	1150	2395	12739
5	18.00	2937	1245	1245	4358



Storey shear of the lateral pattern at the governing step (exact for a force pattern scaled to the base shear)

Pushover Y (2400 yr) (pushover, at the target roof displacement 106 mm (step 30))

Storey	Top z (m)	Weight (kN)	Storey force (kN)	Storey shear (kN)	Overturning at its base (kN·m)
1	4.00	3582	331	4121	53471
2	7.50	3366	584	3790	36988
3	11.00	3366	856	3206	23724
4	14.50	3366	1128	2350	12503
5	18.00	2937	1222	1222	4277



Storey shear of the lateral pattern at the governing step (exact for a force pattern scaled to the base shear)

Time history LS (1000 yr) (time history, suite mean)

Floor z (m)	Peak floor acceleration along / across (g)	Weight (kN)	Σ w·PFA above (kN)
4.00	0.289 / 0.275	3582	5799
7.50	0.306 / 0.299	3366	4764
11.00	0.346 / 0.331	3366	3736
14.50	0.392 / 0.371	3366	2571
18.00	0.426 / 0.420	2937	1252

Σ w·PFA adds the peak floor inertia forces above each floor: an upper bound of the storey shear, the peaks not being simultaneous.

Time history CP (2400 yr) (time history, suite mean)

Floor z (m)	Peak floor acceleration along / across (g)	Weight (kN)	Σ w·PFA above (kN)
4.00	0.354 / 0.402	3582	6753
7.50	0.339 / 0.359	3366	5312
11.00	0.398 / 0.363	3366	4104
14.50	0.419 / 0.399	3366	2764
18.00	0.461 / 0.449	2937	1353

Σ w·PFA adds the peak floor inertia forces above each floor: an upper bound of the storey shear, the peaks not being simultaneous.

Time history OP (50 yr) (time history, suite mean)

Floor z (m)	Peak floor acceleration along / across (g)	Weight (kN)	Σ w·PFA above (kN)
4.00	0.101 / 0.097	3582	2439
7.50	0.135 / 0.127	3366	2077
11.00	0.151 / 0.145	3366	1623
14.50	0.167 / 0.159	3366	1116
18.00	0.189 / 0.183	2937	555

Σ w·PFA adds the peak floor inertia forces above each floor: an upper bound of the storey shear, the peaks not being simultaneous.

No weight irregularity (Table 5.3-2 V-2).

4.3 Member-level plastic deformations

Plastic rotation θp = (φ − φy)·Lh of every yielded member end against the ASCE 41-17 acceptance criteria (Tables 9-7.1, 10-7 to 10-9, 10-19) at the structural level of each study's objective.

Pushover X (2400 yr)

Member level checked: CP (Table 2.4-1 (collapse prevention)); at the target roof displacement 97 mm (step 72). 34 of 330 hinge ends yielded.

Element	End	Member	Section	θp (mrad)	IO	LS	CP	θp / CP	State
21	i	column	C1 storey 1	13.3	3.2	17.7	24.8	0.54	LS
9	i	column	C1 storey 1	13.0	3.4	19.7	27.6	0.47	LS
15	i	column	C1 storey 1	12.8	3.5	20.2	28.3	0.45	LS
22	j	column	C1 storey 1	11.3	3.0	17.8	24.9	0.45	LS
19	i	column	C1 storey 1	13.3	3.8	21.5	30.1	0.44	LS
23	i	column	C1 storey 1	13.3	3.8	21.5	30.1	0.44	LS
16	j	column	C1 storey 1	12.3	3.3	20.3	28.5	0.43	LS
10	j	column	C1 storey 1	11.5	3.2	19.8	27.8	0.41	LS
11	i	column	C1 storey 1	12.4	4.0	24.3	34.1	0.36	LS
7	i	column	C1 storey 1	12.4	4.0	24.3	34.1	0.36	LS
13	i	column	C1 storey 1	12.5	4.1	25.1	35.2	0.36	LS
17	i	column	C1 storey 1	12.5	4.1	25.1	35.2	0.36	LS
24	j	column	C1 storey 1	9.9	3.5	21.6	30.3	0.33	LS
20	j	column	C1 storey 1	9.9	3.5	21.6	30.3	0.33	LS
12	j	column	C1 storey 1	11.1	3.8	24.5	34.3	0.32	LS
8	j	column	C1 storey 1	11.1	3.8	24.5	34.3	0.32	LS
14	j	column	C1 storey 1	11.3	3.9	25.3	35.4	0.32	LS
18	j	column	C1 storey 1	11.3	3.9	25.3	35.4	0.32	LS
3	i	column	C1 storey 1	11.2	5.0	29.0	40.6	0.28	LS

Element	End	Member	Section	θp (mrad)	IO	LS	CP	θp / CP	State
5	i	column	C1 storey 1	10.5	5.0	29.0	40.6	0.26	LS

Pushover Y (2400 yr)

Member level checked: CP (Table 2.4-1 (collapse prevention)); at the target roof displacement 106 mm (step 30). 34 of 330 hinge ends yielded.

Element	End	Member	Section	θp (mrad)	IO	LS	CP	θp / CP	State
17	i	column	C1 storey 1	13.8	3.4	18.6	26.0	0.53	LS
11	i	column	C1 storey 1	13.8	3.3	18.6	26.0	0.53	LS
5	i	column	C1 storey 1	13.8	3.7	20.9	29.3	0.47	LS
23	i	column	C1 storey 1	13.8	3.7	20.9	29.3	0.47	LS
9	i	column	C1 storey 1	13.1	3.5	21.4	30.0	0.44	LS
15	i	column	C1 storey 1	13.1	3.5	21.4	30.0	0.44	LS
10	j	column	C1 storey 1	12.5	3.4	21.5	30.2	0.41	LS
16	j	column	C1 storey 1	12.5	3.4	21.6	30.2	0.41	LS
18	j	column	C1 storey 1	10.6	3.1	18.7	26.1	0.41	LS
12	j	column	C1 storey 1	10.6	3.1	18.7	26.1	0.41	LS
3	i	column	C1 storey 1	12.9	3.9	24.5	34.3	0.38	LS
21	i	column	C1 storey 1	12.8	4.0	24.5	34.3	0.37	LS
4	j	column	C1 storey 1	11.4	3.8	24.6	34.5	0.33	LS
22	j	column	C1 storey 1	11.4	3.8	24.6	34.5	0.33	LS
6	j	column	C1 storey 1	9.4	3.5	21.0	29.4	0.32	LS
24	j	column	C1 storey 1	9.4	3.4	21.0	29.4	0.32	LS
7	i	column	C1 storey 1	11.6	5.0	29.0	40.6	0.29	LS
13	i	column	C1 storey 1	11.6	5.0	29.0	40.6	0.29	LS
1	i	column	C1 storey 1	10.7	5.0	29.0	40.6	0.26	LS
19	i	column	C1 storey 1	10.7	5.0	29.0	40.6	0.26	LS

Time history LS (1000 yr)

Member level checked: LS (Table 2.4-1 (life safety)); over the 14 runs of the record suite (mean response). 53 of 330 hinge ends yielded.

Element	End	Member	Section	θp (mrad)	IO	LS	CP	θp / LS	State
15	i	column	C1 storey 1	4.1	3.3	19.8	27.7	0.21	LS
9	i	column	C1 storey 1	4.1	3.4	20.4	28.6	0.20	LS
3	i	column	C1 storey 1	3.8	3.2	18.9	26.5	0.20	LS
17	i	column	C1 storey 1	3.5	3.5	21.8	30.5	0.16	IO
11	i	column	C1 storey 1	3.5	3.6	22.5	31.6	0.15	IO
5	i	column	C1 storey 1	3.1	3.4	20.6	28.9	0.15	IO
13	i	column	C1 storey 1	3.7	4.1	27.9	39.1	0.13	IO

Element	End	Member	Section	θp (mrad)	IO	LS	CP	θp / LS	State
1	i	column	C1 storey 1	3.4	4.0	25.9	36.2	0.13	IO
7	i	column	C1 storey 1	3.7	4.2	28.9	40.4	0.13	IO
21	i	column	C1 storey 1	3.6	4.7	29.0	40.6	0.12	IO
16	j	column	C1 storey 1	2.2	3.3	19.8	27.8	0.11	IO
19	i	column	C1 storey 1	3.1	5.0	29.0	40.6	0.11	IO
10	j	column	C1 storey 1	2.1	3.4	20.5	28.7	0.10	IO
23	i	column	C1 storey 1	2.9	4.9	29.0	40.6	0.10	IO
4	j	column	C1 storey 1	1.5	3.2	19.0	26.5	0.08	IO
18	j	column	C1 storey 1	1.1	3.6	21.9	30.7	0.05	IO
14	j	column	C1 storey 1	1.4	4.1	28.1	39.3	0.05	IO
12	j	column	C1 storey 1	1.1	3.6	22.6	31.7	0.05	IO
8	j	column	C1 storey 1	1.4	4.2	29.0	40.6	0.05	IO
22	j	column	C1 storey 1	1.3	4.9	29.0	40.6	0.05	IO

Time history CP (2400 yr)

Member level checked: CP (Table 2.4-1 (collapse prevention)); over the 14 runs of the record suite (mean response). 69 of 330 hinge ends yielded.

Element	End	Member	Section	θp (mrad)	IO	LS	CP	θp / CP	State
21	i	column	C1 storey 1	15.0	3.4	18.6	26.0	0.58	LS
23	i	column	C1 storey 1	14.9	3.5	20.1	28.2	0.53	LS
22	j	column	C1 storey 1	13.5	3.3	18.6	26.1	0.52	LS
9	i	column	C1 storey 1	14.8	3.6	20.6	28.8	0.51	LS
15	i	column	C1 storey 1	14.9	3.6	20.9	29.2	0.51	LS
10	j	column	C1 storey 1	14.3	3.6	20.7	29.0	0.49	LS
16	j	column	C1 storey 1	14.4	3.6	21.0	29.4	0.49	LS
11	i	column	C1 storey 1	14.9	3.7	22.2	31.1	0.48	LS
17	i	column	C1 storey 1	15.0	3.7	22.7	31.7	0.47	LS
24	j	column	C1 storey 1	12.0	3.4	20.2	28.3	0.42	LS
19	i	column	C1 storey 1	14.3	3.9	24.7	34.6	0.41	LS
12	j	column	C1 storey 1	13.0	3.6	22.3	31.2	0.41	LS
18	j	column	C1 storey 1	13.1	3.7	22.8	32.0	0.41	LS
7	i	column	C1 storey 1	14.6	4.1	27.8	39.0	0.38	LS
13	i	column	C1 storey 1	14.7	4.2	28.3	39.6	0.37	LS
3	i	column	C1 storey 1	14.7	4.7	29.0	40.6	0.36	LS
5	i	column	C1 storey 1	14.1	4.9	29.0	40.6	0.35	LS
1	i	column	C1 storey 1	14.1	5.0	29.0	40.6	0.35	LS
20	j	column	C1 storey 1	11.4	3.9	24.8	34.8	0.33	LS
8	j	column	C1 storey 1	12.7	4.1	28.0	39.2	0.32	LS

Time history OP (50 yr)

Member level checked: IO (Table 2.4-1 (habitable), §15.4(1)); over the 14 runs of the record suite (mean response). 0 of 330 hinge ends yielded.

No member end yielded.

The limits and coefficients are checked against ASCE/SEI 41-17: steel Table 9-7.1 and Eqs. 9-1 to 9-7; RC beams Table 10-7; RC columns Tables 10-8 and 10-9 with their notes and Eq. 10-3; RC walls Table 10-19; joints Table 10-12 and Eq. 10-4; target displacement Eqs. 7-27 to 7-33 and Tables 7-4 and 7-5. The FEMA 356 coefficients and N2 are checked against published worked examples. Engineering judgement: a roof joint with beams on two opposite faces is classed as exterior; the wall check assumes unconfined boundaries, low axial load and high shear because the piers' forces are not in the results.

4.4 Member-level demand-to-capacity ratios (DCR)

Force-controlled actions (KDS 41 17 00 §1.7(1)–(2): shear and connection failures must be prevented so the members can yield in flexure): member shear against Vc + Vs, beam–column joint shear against ASCE 41-17 Eq. 10-4, and for suites the mean of the largest shears of the runs.

KDS 41 17 00 Chapter 15 gives no amplification of force-controlled demands (1.2 × the mean is asked only of damping-system components, §17.2.2(1)②): the mean of the suite is compared with the nominal capacity Vc + Vs without a strength reduction factor. Whether a margin (for example 1.2 to 1.5 on the mean, as ASCE 7 §16.4.2 and the tall-building guidelines ask) should apply is a point for the review.

Pushover X (2400 yr)

Action	Where	Demand (kN)	Capacity (kN)	DCR
Shear	element 126 end i (C1 storey 1)	551	771	0.72
Shear	element 127 end j (C1 storey 1)	549	771	0.71
Shear	element 120 end i (C1 storey 1)	543	771	0.70
Shear	element 121 end j (C1 storey 1)	540	771	0.70
Shear	element 60 end j (B4 400×700 T 1750×200)	239	350	0.68
Shear	element 21 end i (C1 storey 1)	515	771	0.67
Shear	element 22 end j (C1 storey 1)	515	771	0.67
Shear	element 9 end i (C1 storey 1)	512	771	0.66
Shear	element 10 end j (C1 storey 1)	512	771	0.66
Shear	element 15 end i (C1 storey 1)	503	771	0.65
Joint shear	node 20 (exterior with transverse beams)	1096	2372	0.46
Joint shear	node 24 (exterior with transverse beams)	1096	2372	0.46
Joint shear	node 22 (interior with transverse beams)	1203	3162	0.38
Joint shear	node 8 (interior with transverse beams)	1189	3162	0.38
Joint shear	node 12 (interior with transverse beams)	1189	3162	0.38
Joint shear	node 10 (interior with transverse beams)	1171	3162	0.37
Joint shear	node 14 (interior with transverse beams)	1151	3162	0.36
Joint shear	node 18 (interior with transverse beams)	1151	3162	0.36

Pushover Y (2400 yr)

Action	Where	Demand (kN)	Capacity (kN)	DCR
Shear	element 126 end i (C1 storey 1)	576	771	0.75
Shear	element 120 end i (C1 storey 1)	576	771	0.75
Shear	element 127 end j (C1 storey 1)	573	771	0.74
Shear	element 121 end j (C1 storey 1)	573	771	0.74
Shear	element 114 end i (C1 storey 1)	514	771	0.67
Shear	element 132 end i (C1 storey 1)	514	771	0.67
Shear	element 115 end j (C1 storey 1)	512	771	0.66
Shear	element 133 end j (C1 storey 1)	512	771	0.66
Shear	element 15 end i (C1 storey 1)	506	771	0.66
Shear	element 9 end i (C1 storey 1)	506	771	0.66
Joint shear	node 24 (exterior with transverse beams)	872	2372	0.37
Joint shear	node 6 (exterior with transverse beams)	872	2372	0.37
Joint shear	node 4 (interior with transverse beams)	1134	3162	0.36
Joint shear	node 22 (interior with transverse beams)	1134	3162	0.36
Joint shear	node 43 (exterior with transverse beams)	724	2372	0.31
Joint shear	node 52 (exterior with transverse beams)	724	2372	0.31
Joint shear	node 42 (interior with transverse beams)	949	3162	0.30
Joint shear	node 51 (interior with transverse beams)	948	3162	0.30

Time history LS (1000 yr)

Action	Where	Demand (kN)	Capacity (kN)	DCR
Shear (shown run)	element 126 end i (C1 storey 1)	543	771	0.70
Shear (shown run)	element 120 end i (C1 storey 1)	541	771	0.70
Shear (shown run)	element 127 end j (C1 storey 1)	540	771	0.70
Shear (shown run)	element 121 end j (C1 storey 1)	538	771	0.70
Shear, mean of the suite	element 120 end i	532	771	0.69
Shear, mean of the suite	element 126 end i	531	771	0.69
Shear, mean of the suite	element 121 end j	529	771	0.69
Shear, mean of the suite	element 127 end j	529	771	0.69
Shear (shown run)	element 3 end i (C1 storey 1)	489	771	0.63
Shear (shown run)	element 15 end i (C1 storey 1)	488	771	0.63
Shear (shown run)	element 4 end j (C1 storey 1)	487	771	0.63
Shear (shown run)	element 16 end j (C1 storey 1)	486	771	0.63
Shear (shown run)	element 132 end i (C1 storey 1)	483	771	0.63
Shear (shown run)	element 133 end j (C1 storey 1)	481	771	0.62
Shear, mean of the suite	element 16 end j	479	771	0.62
Shear, mean of the suite	element 15 end i	478	771	0.62

Action	Where	Demand (kN)	Capacity (kN)	DCR
Shear, mean of the suite	element 9 end i	478	771	0.62
Shear, mean of the suite	element 10 end j	477	771	0.62
Joint shear	node 6 (exterior with transverse beams)	1209	2372	0.51
Joint shear	node 4 (interior with transverse beams)	1475	3162	0.47
Joint shear	node 2 (exterior with transverse beams)	1029	2372	0.43
Joint shear	node 18 (interior with transverse beams)	1340	3162	0.42
Joint shear	node 16 (interior with transverse beams)	1324	3162	0.42
Joint shear	node 12 (interior with transverse beams)	1304	3162	0.41
Joint shear	node 10 (interior with transverse beams)	1280	3162	0.40
Joint shear	node 43 (exterior with transverse beams)	946	2372	0.40

Time history CP (2400 yr)

Action	Where	Demand (kN)	Capacity (kN)	DCR
Shear, mean of the suite	element 126 end i	551	771	0.71
Shear, mean of the suite	element 120 end i	549	771	0.71
Shear, mean of the suite	element 127 end j	549	771	0.71
Shear, mean of the suite	element 121 end j	547	771	0.71
Shear (shown run)	element 120 end i (C1 storey 1)	542	771	0.70
Shear (shown run)	element 121 end j (C1 storey 1)	539	771	0.70
Shear (shown run)	element 126 end i (C1 storey 1)	539	771	0.70
Shear (shown run)	element 127 end j (C1 storey 1)	536	771	0.70
Shear, mean of the suite	element 132 end i	505	771	0.66
Shear, mean of the suite	element 133 end j	503	771	0.65
Shear, mean of the suite	element 10 end j	492	771	0.64
Shear, mean of the suite	element 9 end i	491	771	0.64
Shear (shown run)	element 124 end i (C1 storey 1)	489	771	0.63
Shear (shown run)	element 114 end i (C1 storey 1)	488	771	0.63
Shear (shown run)	element 15 end i (C1 storey 1)	488	771	0.63
Shear (shown run)	element 16 end j (C1 storey 1)	487	771	0.63
Shear (shown run)	element 3 end i (C1 storey 1)	486	771	0.63
Shear (shown run)	element 125 end j (C1 storey 1)	486	771	0.63
Joint shear	node 24 (exterior with transverse beams)	1320	2372	0.56
Joint shear	node 22 (interior with transverse beams)	1513	3162	0.48
Joint shear	node 20 (exterior with transverse beams)	1127	2372	0.48
Joint shear	node 12 (interior with transverse beams)	1373	3162	0.43
Joint shear	node 18 (interior with transverse beams)	1344	3162	0.42
Joint shear	node 52 (exterior with transverse beams)	987	2372	0.42
Joint shear	node 10 (interior with transverse beams)	1272	3162	0.40

Action	Where	Demand (kN)	Capacity (kN)	DCR
Joint shear	node 16 (interior with transverse beams)	1246	3162	0.39

Time history OP (50 yr)

Action	Where	Demand (kN)	Capacity (kN)	DCR
Shear (shown run)	element 126 end i (C1 storey 1)	290	771	0.38
Shear (shown run)	element 127 end j (C1 storey 1)	289	771	0.37
Shear (shown run)	element 120 end i (C1 storey 1)	286	771	0.37
Shear (shown run)	element 60 end j (B4 400×700 T 1750×200)	130	350	0.37
Shear (shown run)	element 121 end j (C1 storey 1)	285	771	0.37
Shear (shown run)	element 52 end i (B4 400×700 T 1750×200)	126	350	0.36
Shear, mean of the suite	element 60 end j	124	350	0.36
Shear (shown run)	element 9 end i (C1 storey 1)	273	771	0.35
Shear (shown run)	element 10 end j (C1 storey 1)	273	771	0.35
Shear (shown run)	element 15 end i (C1 storey 1)	273	771	0.35
Shear (shown run)	element 16 end j (C1 storey 1)	273	771	0.35
Shear, mean of the suite	element 52 end i	123	350	0.35
Shear, mean of the suite	element 126 end i	257	771	0.33
Shear, mean of the suite	element 120 end i	256	771	0.33
Shear, mean of the suite	element 127 end j	255	771	0.33
Shear, mean of the suite	element 121 end j	254	771	0.33
Shear, mean of the suite	element 171 end j	112	350	0.32
Shear, mean of the suite	element 163 end i	111	350	0.32
Joint shear	node 24 (exterior with transverse beams)	340	2372	0.14
Joint shear	node 16 (interior with transverse beams)	445	3162	0.14
Joint shear	node 10 (interior with transverse beams)	442	3162	0.14
Joint shear	node 52 (exterior with transverse beams)	327	2372	0.14
Joint shear	node 18 (interior with transverse beams)	403	3162	0.13
Joint shear	node 48 (interior with transverse beams)	402	3162	0.13
Joint shear	node 12 (interior with transverse beams)	400	3162	0.13
Joint shear	node 22 (interior with transverse beams)	395	3162	0.12

4.5 Performance level assessment (IO / LS / CP)

Each study against the objective of its return period: storey drift against the drift limit, member plastic rotations against the structural level (largest θp / limit), force-controlled actions (largest DCR), and no collapse.

Study	Objective	Drift / limit	Members θp / limit	Force-controlled DCR	Worst hinge	Result
Pushover X (2400 yr)	Collapse prevention at 2400 years	0.54 % / 3.00 %	0.54	0.72	LS	MET

Study	Objective	Drift / limit	Members θp / limit	Force-controlled DCR	Worst hinge	Result
Pushover Y (2400 yr)	Collapse prevention at 2400 years	0.60 % / 3.00 %	0.53	0.75	LS	MET
Time history LS (1000 yr)	Life safety at 1000 years	0.69 % / 2.00 %	0.21	0.70	LS	MET
Time history CP (2400 yr)	Collapse prevention at 2400 years	1.59 % / 3.00 %	0.58	0.71	LS	MET
Time history OP (50 yr)	Operational at 50 years	0.12 % / 0.50 %	0.00 (may be omitted)	0.38	Elastic	MET

Objectives met: 3 (2400 years, 1000 years, 50 years); §15.3.1 asks for at least 2 of Table 15.3-1.

PERFORMANCE OBJECTIVES MET

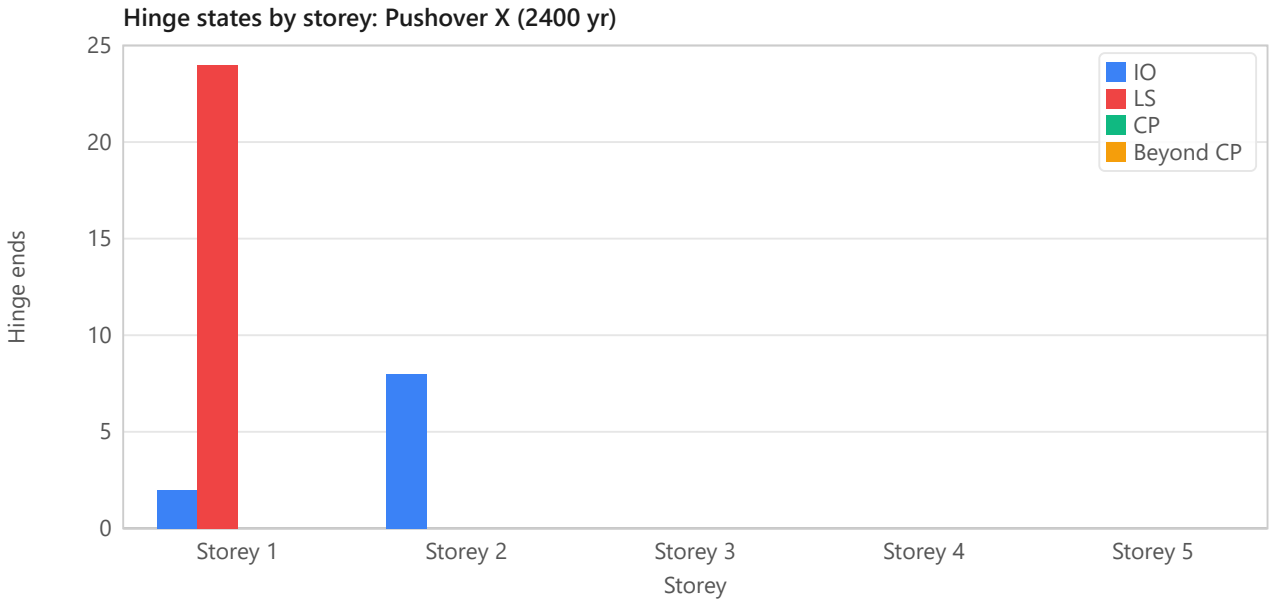
4.6 Damage distribution

Where the yielding is: the hinge ends of each storey by the level they reach and by member type. The 3D view shows them on the model (Results › Seismic › hinges).

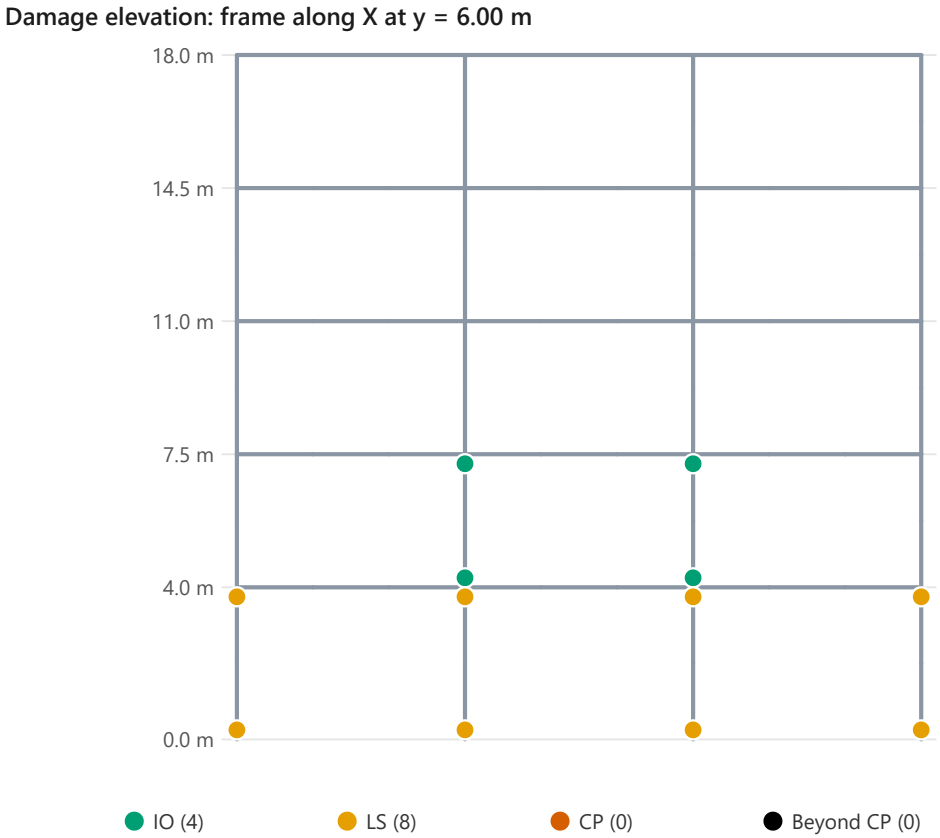
Pushover X (2400 yr)

at the target roof displacement 97 mm (step 72): 296 elastic, 10 IO, 24 LS, 0 CP, 0 beyond CP hinge ends. Column hinges outnumber beam hinges in storey 1, 2: check the strong-column / weak-beam intent (KDS 41 17 00 §1.7(3)) and a storey mechanism.

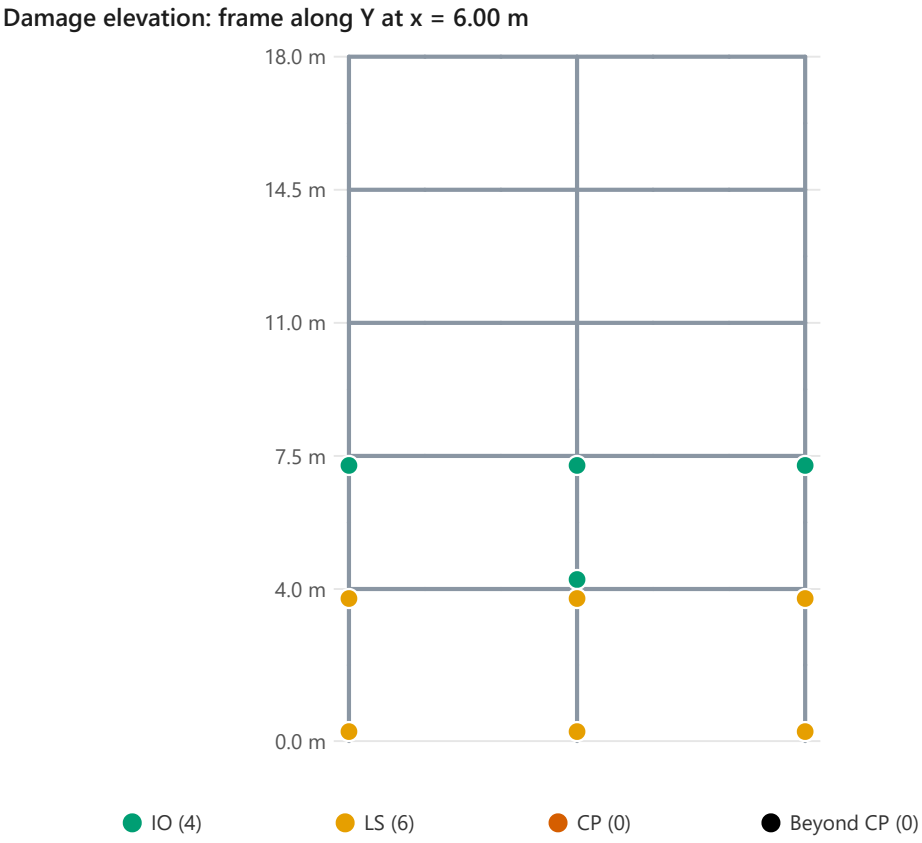
Storey	IO	LS	CP	Beyond CP	Beam ends	Column ends	Steel ends
1	2	24	0	0	2	24	0
2	8	0	0	0	0	8	0
3	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0



Yielded hinge ends of each storey by the performance level they reach



Pushover X (2400 yr): yielded hinge ends on the most damaged frame along X at y = 6.00 m, at the target roof displacement 97 mm (step 72), coloured by the level they reach

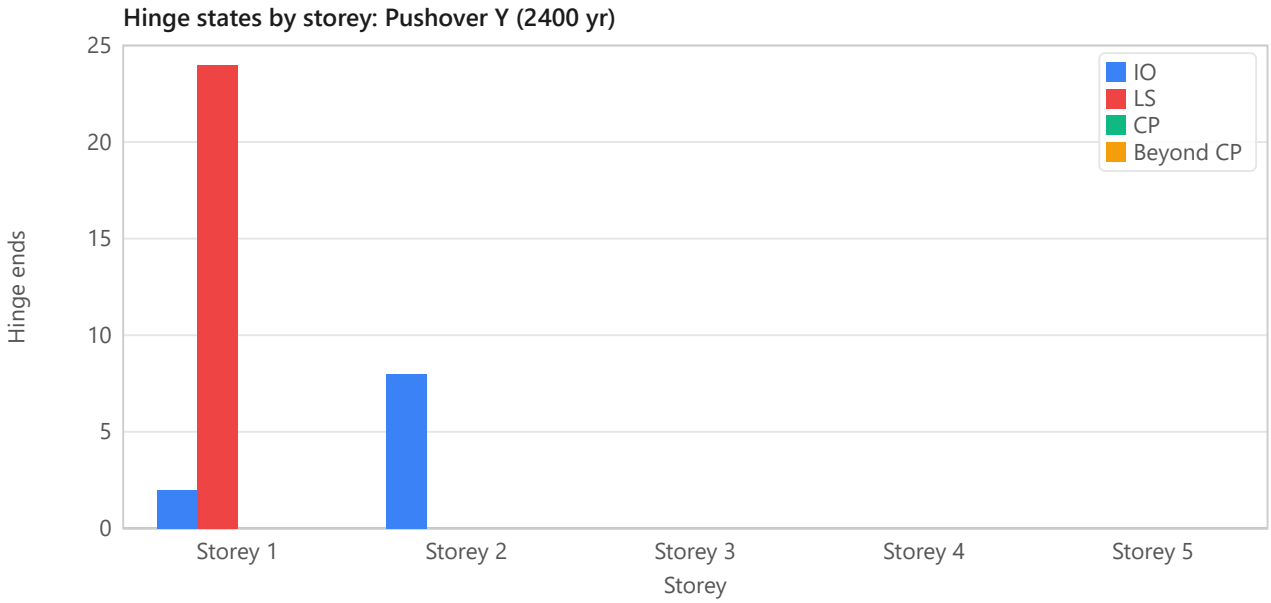


Pushover X (2400 yr): yielded hinge ends on the most damaged frame along Y at x = 6.00 m, at the target roof displacement 97 mm (step 72), coloured by the level they reach

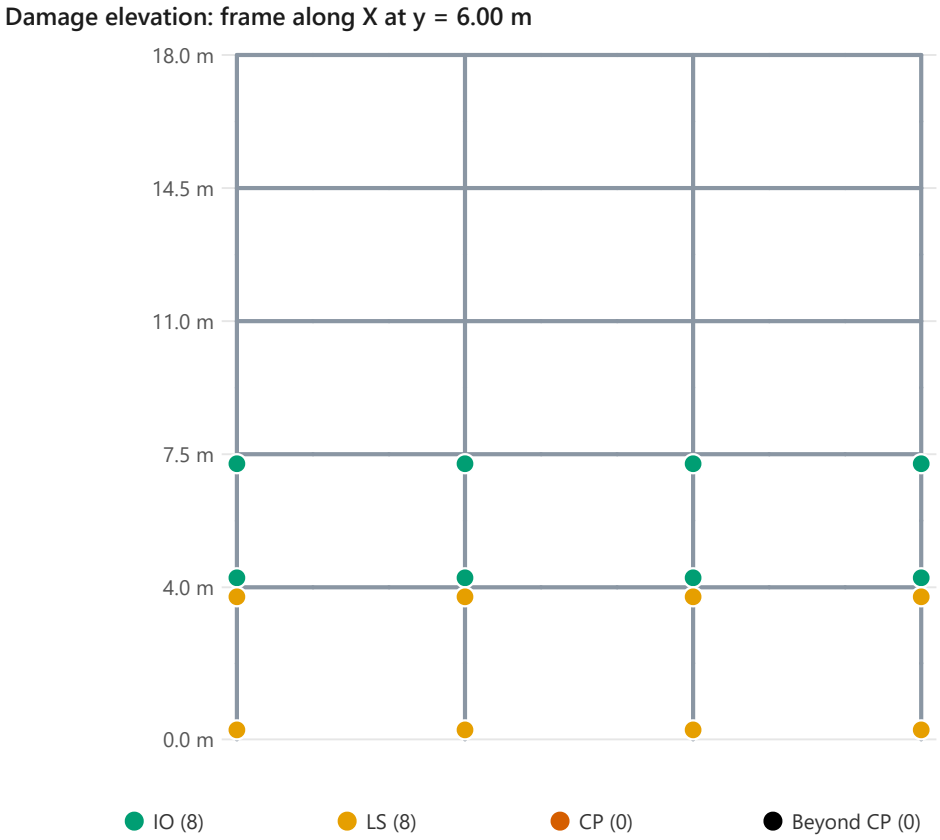
Pushover Y (2400 yr)

at the target roof displacement 106 mm (step 30): 296 elastic, 10 IO, 24 LS, 0 CP, 0 beyond CP hinge ends. Column hinges outnumber beam hinges in storey 1, 2: check the strong-column / weak-beam intent (KDS 41 17 00 §1.7(3)) and a storey mechanism.

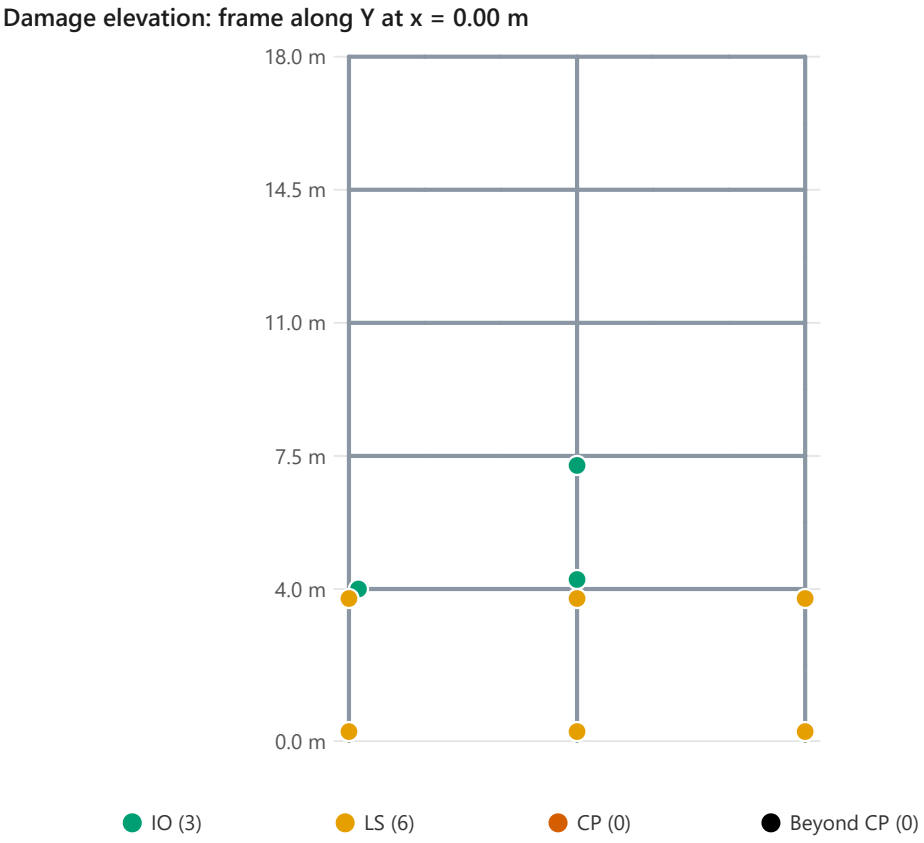
Storey	IO	LS	CP	Beyond CP	Beam ends	Column ends	Steel ends
1	2	24	0	0	2	24	0
2	8	0	0	0	0	8	0
3	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0



Yielded hinge ends of each storey by the performance level they reach



Pushover Y (2400 yr): yielded hinge ends on the most damaged frame along X at y = 6.00 m, at the target roof displacement 106 mm (step 30), coloured by the level they reach



Pushover Y (2400 yr): yielded hinge ends on the most damaged frame along Y at x = 0.00 m, at the target roof displacement 106 mm (step 30), coloured by the level they reach

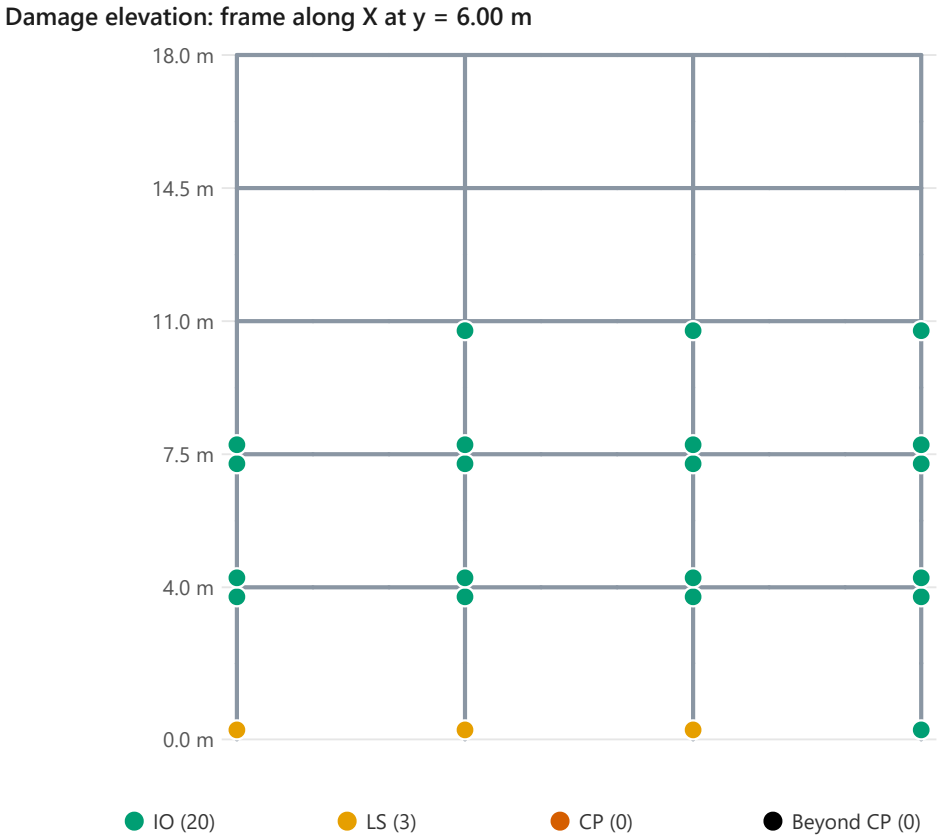
Time history LS (1000 yr)

over the 14 runs of the record suite (mean response): 277 elastic, 50 IO, 3 LS, 0 CP, 0 beyond CP hinge ends. Column hinges outnumber beam hinges in storey 1, 2, 3: check the strong-column / weak-beam intent (KDS 41 17 00 §1.7(3)) and a storey mechanism.

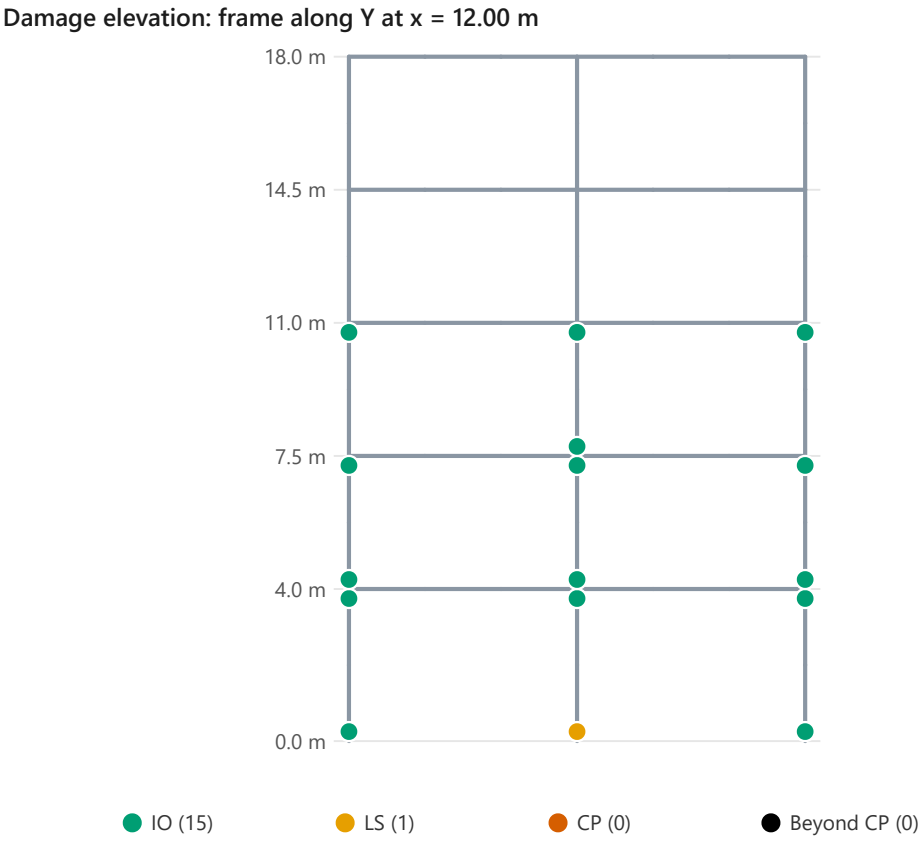
Storey	IO	LS	CP	Beyond CP	Beam ends	Column ends	Steel ends
1	24	3	0	0	3	24	0
2	16	0	0	0	0	16	0
3	10	0	0	0	0	10	0
4	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0



Yielded hinge ends of each storey by the performance level they reach



Time history LS (1000 yr): yielded hinge ends on the most damaged frame along X at y = 6.00 m, over the 14 runs of the record suite (mean response), coloured by the level they reach

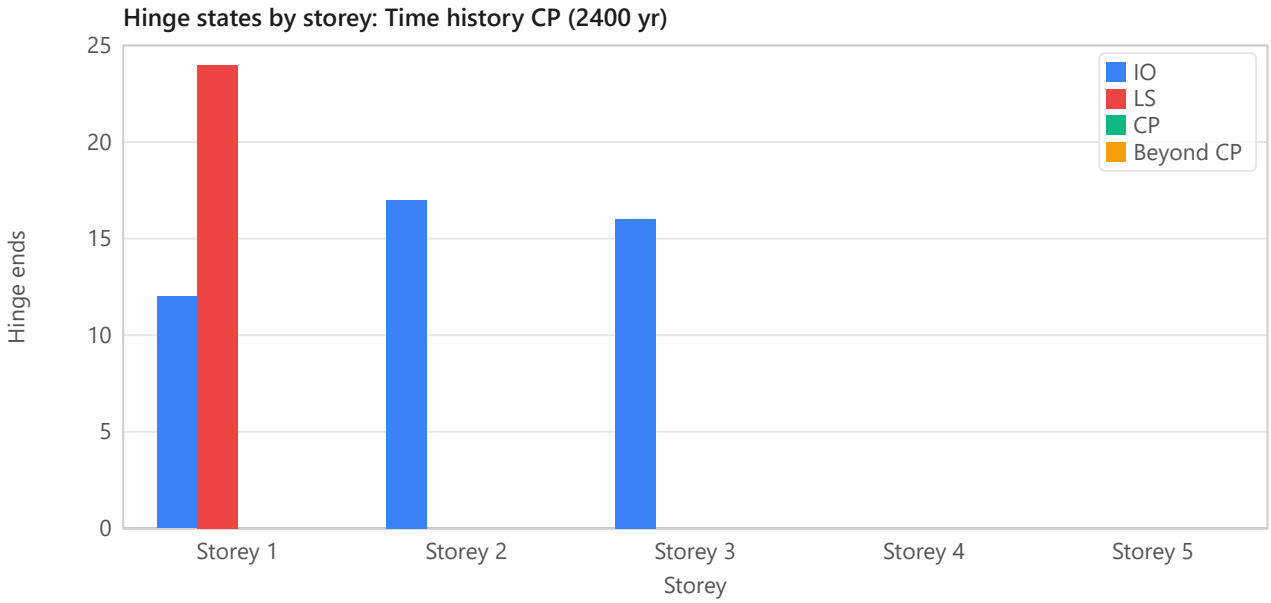


Time history LS (1000 yr): yielded hinge ends on the most damaged frame along Y at x = 12.00 m, over the 14 runs of the record suite (mean response), coloured by the level they reach

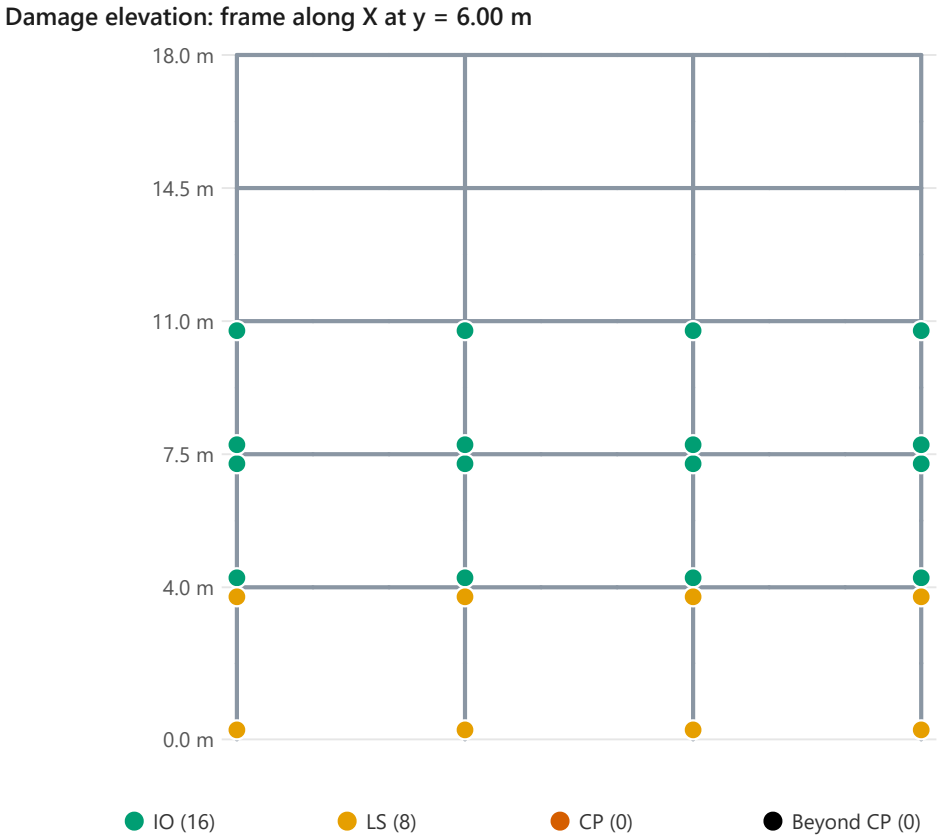
Time history CP (2400 yr)

over the 14 runs of the record suite (mean response): 261 elastic, 45 IO, 24 LS, 0 CP, 0 beyond CP hinge ends. Column hinges outnumber beam hinges in storey 1, 2, 3: check the strong-column / weak-beam intent (KDS 41 17 00 §1.7(3)) and a storey mechanism.

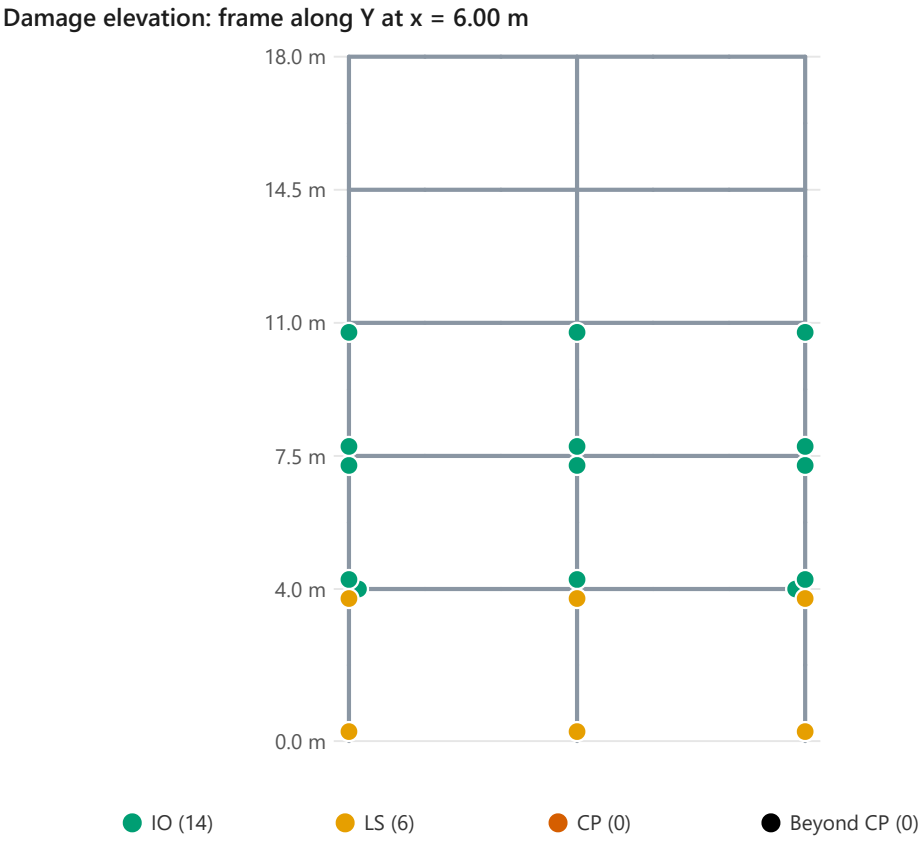
Storey	IO	LS	CP	Beyond CP	Beam ends	Column ends	Steel ends
1	12	24	0	0	12	24	0
2	17	0	0	0	0	17	0
3	16	0	0	0	0	16	0
4	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0



Yielded hinge ends of each storey by the performance level they reach



Time history CP (2400 yr): yielded hinge ends on the most damaged frame along X at y = 6.00 m, over the 14 runs of the record suite (mean response), coloured by the level they reach

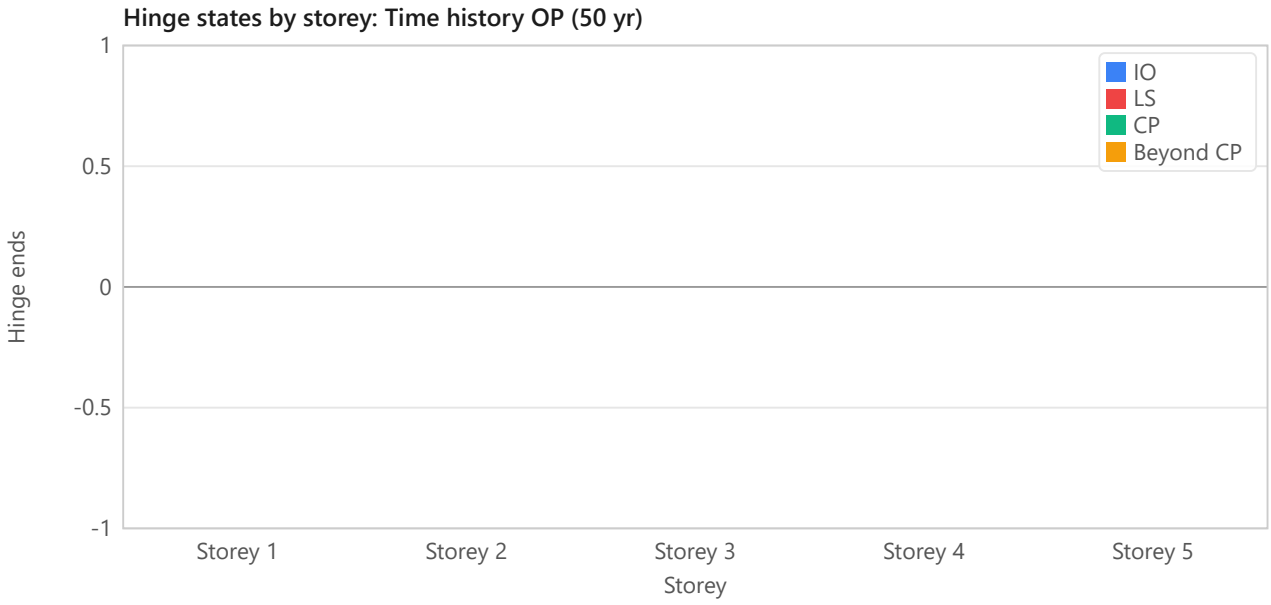


Time history CP (2400 yr): yielded hinge ends on the most damaged frame along Y at x = 6.00 m, over the 14 runs of the record suite (mean response), coloured by the level they reach

Time history OP (50 yr)

over the 14 runs of the record suite (mean response): 330 elastic, 0 IO, 0 LS, 0 CP, 0 beyond CP hinge ends. The yielding is in the beams more than the columns (strong column / weak beam, KDS 41 17 00 §1.7(3)).

Storey	IO	LS	CP	Beyond CP	Beam ends	Column ends	Steel ends
1	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0
3	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0



Yielded hinge ends of each storey by the performance level they reach

5 Comprehensive review and conclusions

Verification of the analyses, the summary, the independent review and the conclusions.

5.1 Verification and validation of analysis results

Item	Status
KDS 41 17 00 : 2022 values (site coefficients, spectrum, Cs, Cu, objectives, drift limits, record rules, irregularities)	checked against the 2022 text; unit tests reproduce the printed tables
KDS zone factors Z (KDS 17 10 00)	from KDS 17 10 00 Table 4.2-2 — not in KDS 41 17 00: verify
ASCE 41-17 acceptance criteria and target displacement	checked against ASCE/SEI 41-17 (Tables 9-7.1, 10-7 to 10-9, 10-12, 10-19; Eqs. 7-27 to 7-33)
Coefficient method / N2	reproduce published worked examples (FEMA 356, EN 1998-1 Annex B)
OpenSees model vs the linear solver	static and modal results of the same models agree (beams, trusses, solids to ~1e-9; shells 0.1–2 %)
Single RC column pushover	within 3 % of the ACI flexural strength
Equilibrium and convergence	converged steps only are reported; non-converged pushover steps are rejected
Energy balance of the time histories (input = kinetic + damping + restoring)	Time history LS (1000 yr): largest error 0.46 % of the input; Time history CP (2400 yr): largest error 0.33 % of the input; Time history OP (50 yr): largest error 0.59 % of the input
Runs stopped early	none
Benchmark against other software (Perform3D, MIDAS, ETABS) on this building	to be done by the engineer if the review asks for it

The limits and coefficients are checked against ASCE/SEI 41-17: steel Table 9-7.1 and Eqs. 9-1 to 9-7; RC beams Table 10-7; RC columns Tables 10-8 and 10-9 with their notes and Eq. 10-3; RC walls Table 10-19; joints Table 10-12 and Eq. 10-4; target displacement Eqs. 7-27 to 7-33 and Tables 7-4 and 7-5. The FEMA 356 coefficients and N2 are checked against published worked examples. Engineering judgement: a roof joint with beams on two opposite faces is classed as exterior; the wall check assumes unconfined boundaries, low axial load and high shear because the piers' forces are not in the results.

The comparison of the nonlinear with the linear analyses that §15.5 ④ asks for is in the linear analysis verification (Part 3).

5.2 Summary of seismic performance evaluation

Study	Finding	Status
Pushover X (2400 yr)	ASCE 41-17 target 88 mm (0.49 %): worst hinge LS.	OK
Pushover X (2400 yr)	N2 target 97 mm (0.54 %): worst hinge LS.	OK
Pushover Y (2400 yr)	ASCE 41-17 target 94 mm (0.52 %): worst hinge LS.	OK
Pushover Y (2400 yr)	N2 target 106 mm (0.59 %): worst hinge LS.	OK
Time history LS (1000 yr)	Peak storey drift 0.60 %, residual roof drift 0.00 %, worst hinge IO.	OK

Study	Finding	Status
Time history CP (2400 yr)	Peak storey drift 1.19 %, residual roof drift 0.04 %, worst hinge LS.	OK
Time history OP (50 yr)	Peak storey drift 0.12 %, residual roof drift 0.00 %, worst hinge Elastic.	OK
Performance objectives	3 of the 3 objectives of Table 15.3-1 (grade II) have a study; §15.3.1 asks for at least two	OK
Linear verification	Yield base shear 4116 kN against 75 % of the equivalent static 1657 kN (§15.6, as an indicator)	OK
Pushover X (2400 yr)	Roof drift at the target (a pushover gives no storey drifts) 0.54 % against 3.00 % (§15.4(3): 3 % / IE)	OK
Pushover Y (2400 yr)	Roof drift at the target (a pushover gives no storey drifts) 0.60 % against 3.00 % (§15.4(3): 3 % / IE)	OK
Time history LS (1000 yr)	Mean peak storey drift of the suite 0.69 % against 2.00 % (Table 8.2-1 (§7.3.4.3, §8.2.3))	OK
Time history CP (2400 yr)	Mean peak storey drift of the suite 1.59 % against 3.00 % (§15.4(3): 3 % / IE)	OK
Time history OP (50 yr)	Mean peak storey drift of the suite 0.12 % against 0.50 % (§15.4(2))	OK
Pushover X (2400 yr)	Plastic rotations at the target roof displacement 97 mm (step 72): 34 hinge ends yielded, 0 beyond CP (Table 2.4-1 (collapse prevention))	OK
Pushover Y (2400 yr)	Plastic rotations at the target roof displacement 106 mm (step 30): 34 hinge ends yielded, 0 beyond CP (Table 2.4-1 (collapse prevention))	OK
Time history LS (1000 yr)	Plastic rotations over the 14 runs of the record suite (mean response): 53 hinge ends yielded, 0 beyond LS (Table 2.4-1 (life safety))	OK
Time history CP (2400 yr)	Plastic rotations over the 14 runs of the record suite (mean response): 69 hinge ends yielded, 0 beyond CP (Table 2.4-1 (collapse prevention))	OK
Time history OP (50 yr)	Plastic rotations over the 14 runs of the record suite (mean response): 0 hinge ends yielded, 0 beyond IO (Table 2.4-1 (habitable), §15.4(1)) (members may be omitted for the operational check, §15.4(1))	—
Pushover X (2400 yr)	Largest demand-to-capacity ratio of the force-controlled actions 0.72 at the target roof displacement 97 mm (step 72)	OK
Pushover Y (2400 yr)	Largest demand-to-capacity ratio of the force-controlled actions 0.75 at the target roof displacement 106 mm (step 30)	OK
Time history LS (1000 yr)	Largest demand-to-capacity ratio of the force-controlled actions 0.70 over the 14 runs of the record suite (mean response)	OK
Time history CP (2400 yr)	Largest demand-to-capacity ratio of the force-controlled actions 0.71 over the 14 runs of the record suite (mean response)	OK
Time history OP (50 yr)	Largest demand-to-capacity ratio of the force-controlled actions 0.38 over the 14 runs of the record suite (mean response)	OK
Pushover X (2400 yr)	Collapse prevention at 2400 years: met (drift 0.54 % / 3.00 %, members 0.54, force-controlled 0.72)	OK
Pushover Y (2400 yr)	Collapse prevention at 2400 years: met (drift 0.60 % / 3.00 %, members 0.53, force-controlled 0.75)	OK
Time history LS (1000 yr)	Life safety at 1000 years: met (drift 0.69 % / 2.00 %, members 0.21, force-controlled 0.70)	OK
Time history CP (2400 yr)	Collapse prevention at 2400 years: met (drift 1.59 % / 3.00 %, members 0.58, force-controlled 0.71)	OK

Study	Finding	Status
Time history OP (50 yr)	Operational at 50 years: met (drift 0.12 % / 0.50 %, members 0.00, force-controlled 0.38)	OK
Performance objectives	3 of the 3 objectives of Table 15.3-1 met (2400 years, 1000 years, 50 years); §15.3.1 asks for at least two	OK

5.3 Structural engineer's review and comments

KDS 41 17 00 §15.7: the procedure, its basis and the results of a performance-based design are checked by at least 2 third-party reviewers other than the designer — Professional Engineer in building structures (건축구조기술사) with practice in the field, and a PhD with research in the field (§15.7).

Reviewer	Qualification	Organisation	Date	Signature
_____	_____	_____	_____	_____
_____	_____	_____	_____	_____

Points for the reviewer (assumptions that carry the result):

- Expected material strengths and their factors
- The seismic weight (the load combination of the studies)
- Damping ratio and Rayleigh modes
- Zone factor / map value S and site class
- The records and their scaling (§7.3.4.1)
- Joint rigid zones, bar slip and shear modelling
- Wall boundary confinement assumed in the wall acceptance
- Members whose acceptance is engineering judgement (roof joints, walls)

#	Reviewer comment	Designer response / resolution
1	_____	_____
2	_____	_____
3	_____	_____

5.4 Final conclusions and appendices

Every check passed: the building meets the performance objectives evaluated, as analysed and subject to the review and to the points below.

Points to note (not failed checks):

- Time history LS (1000 yr): the drift of storey 1, 3 is more than 130 % of the storey above (0.69 % against 0.43 %; 0.34 % against 0.23 %): the exemption of KDS 41 17 00 §5.3.2(1)① from the stiffness irregularity V-1 does not apply (a nonlinear response, not the elastic design analysis the rule is written for) — check a soft storey in the design model
- Pushover X (2400 yr): the capacity curve ends at 1.20 × the target displacement (strength down to 80 % of the peak); ASCE 41-17 §7.4.3.2.1 asks for the curve up to 150 % of the target
- Pushover Y (2400 yr): the capacity curve ends at 1.26 × the target displacement (strength down to 80 % of the peak); ASCE 41-17 §7.4.3.2.1 asks for the curve up to 150 % of the target
- Pushover X (2400 yr): column hinges outnumber beam hinges in storey 1, 2 (24 column against 2 beam ends; 8 column against 0 beam ends): a storey (column-sway) mechanism, against the strong-column / weak-beam

intent of KDS 41 17 00 §1.7(3)

- Pushover Y (2400 yr): column hinges outnumber beam hinges in storey 1, 2 (24 column against 2 beam ends; 8 column against 0 beam ends): a storey (column-sway) mechanism, against the strong-column / weak-beam intent of KDS 41 17 00 §1.7(3)
- Time history LS (1000 yr): column hinges outnumber beam hinges in storey 1, 2, 3 (24 column against 3 beam ends; 16 column against 0 beam ends; 10 column against 0 beam ends): a storey (column-sway) mechanism, against the strong-column / weak-beam intent of KDS 41 17 00 §1.7(3)
- Time history CP (2400 yr): column hinges outnumber beam hinges in storey 1, 2, 3 (24 column against 12 beam ends; 17 column against 0 beam ends; 16 column against 0 beam ends): a storey (column-sway) mechanism, against the strong-column / weak-beam intent of KDS 41 17 00 §1.7(3)

For the reviewer: (1) are the ASCE 41-17 acceptance criteria acceptable as member limits, KDS 41 17 00 giving none; (2) should force-controlled demands carry a margin on the mean of the suite; (3) is the layout and content of this report what a performance-based design review expects, and what is missing.

References: KDS 41 17 00 : 2022 건축물 내진설계기준; KDS 17 10 00 내진설계 일반; ASCE/SEI 41-17; ASCE/SEI 7-16 / 7-22; FEMA 356; EN 1998-1 Annex B; OpenSees (PEER, UC Berkeley).

5.4.1 Appendix A — study results by study

The full pushover and time history chapters are in Part 3; every table and figure there is part of this appendix.