

Test Plan and Preliminary Analysis of Shake-Table Tests of a Full-Scale
Three-Story Steel Braced Moment-Resisting Frame
実大三層ブレース付ラーメン鋼架構の振動台実験の実験計画と事前解析

ROJAS PALACIOS Julio Cesar

Division of Architecture and Structural Design

Laboratory of Structural Engineering, Research Group of Structural and Urban Safety Design

Abstract

This study presents the design, test plan, and pre-test numerical analysis of a full-scale three-story steel Chevron Braced Moment-Resisting Frame for shake-table testing. The design basis is consistent with Japanese practice, including member proportioning, connection detailing, and capacity-design checks. Required lateral strength and ductility are verified against Level-1 and Level-2 seismic demands. A detailed nonlinear OpenSees model captures brace buckling, panel-zone deformation, and connection flexibility. Pushover and nonlinear time-history analyses quantify expected strength, drift, force redistribution, and stiffness degradation, providing guidance for specimen detailing.

Keywords: Steel structure, Chevron-braced Moment Resisting Frame, Shake-table test, Numerical analysis

1. Introduction

Concentrically Braced Frames are widely used in seismic regions because they provide high lateral stiffness and strength with efficient material use. Among them, Chevron Braced Moment-Resisting Frames (CB-MRF) are common in Japan, where fabrication practices and design decisions often rely on engineering judgment rather than fully prescriptive seismic provisions. This situation highlights the need for well-documented full-scale experimental studies that clarify global behavior, failure mechanisms, and force redistribution under strong ground motion. Shake-table testing offers a unique opportunity to observe realistic dynamic response, including brace buckling, beam demands induced by unbalanced forces, panel-zone deformation, and progressive stiffness degradation.

This thesis is part of an ongoing research effort at Hokkaido University aimed at establishing a rational experimental framework for large-scale steel braced moment-resisting frames. A full-scale three-story Chevron Braced Moment-Resisting Frame is designed to be representative of typical Japanese practice while remaining compatible with laboratory capacity. The study integrates code-based design checks, capacity-design principles, and detailed numerical modeling to define the specimen, the test plan, and the expected

seismic response. The resulting framework supports safe and efficient shake-table testing and provides a reference.

2. Literature review

Previous experimental research provides key insight into the seismic behavior of steel braced frames. Mamani (2025) [1] tested a single-bay, single-story chevron braced moment-resisting frame under cyclic lateral loading at Hokkaido University. The specimen, representing typical Japanese practice, used square tubular columns, H-shaped beams, and circular hollow section braces. Braces buckled early in compression yet maintained stable, symmetric hysteresis up to large drift demands, reaching a maximum drift ratio of 0.05 rad. No fracture or weld cracking was observed in gusset connections, and measured strengths agreed with analytical predictions at brace buckling.

Complementary evidence is provided by Chou et al. (2024) [2], who conducted shake-table tests on a full-scale three-story steel dual frame incorporating buckling-restrained braces. Tests using recorded earthquake motions achieved interstory drift ratios up to 0.045 rad. A sliding-slab configuration reduced roof accelerations while preserving strength and stiffness, and built-up box columns exhibited stable hysteretic behavior without local buckling.

3. Test Plan

Fig. 1 shows the configuration of three-story, single-bay chevron-braced moment-resisting frame specimen. The details of the structural members are listed in Table 1. The braces, connected to the beams and columns by 9.0-mm gusset plates with bolted connections, are aligned so that their centerlines either passed through a beam-to-column node or intersected at the beam's midpoint. The ultimate plastic strength of the combined system, H_2 , is the resulting from the sum of the strengths of the MRF, H_{2f} , and the braces, H_{2b} . A key parameter of design is the proportion of the plastic lateral strength provided by the braces relative to the total frame, defined as $\beta_0 = H_{2b} / H_2$. The parameter κ represents the assumed limit for the maximum unbalanced vertical force between braces over the plastic resistance of the beam. The specimen was designed for a $\frac{(N_y - 0.3N_{cr})\sin(\theta)}{\frac{4M_p}{L}} = 1.5$ proportion the

beams to yield after brace buckling. The specimen was design using plastic analysis based on the kinematic (virtual work) method.

The expected yielding strength was used to compute the plastic strength of the MRF, $H_{2f} = 258$ kN, obtained from the plastic analysis based on the lateral load distribution pattern, A_i , of the Japan Building Law. The computed plastic strength of the braces is $H_{2b} = 430$ kN, while the total strength of the specimen at ultimate plastic mechanism is $H_2 = 688$ kN with the proportion key $\beta_0 = 0.63$.

4. Numerical Model

OpenSees simulation platform [3] was used to conduct numerical simulation. For the two-dimensional analysis, the beams and columns were modeled using force-based elements with five Gauss-Lobatto integration points, fiber discretization with a 2×10 mesh applied to the flanges and 1×10 for webs. Following the recommendation by Karamanci and Lignos [4], the braces were modeled using displacement-based fiber elements with a 12×4 mesh for the circular section. Each brace was divided into eight elements with five Gauss-Lobatto integration points. Nonlinear geometry was considered using corotational transformation. The Menegotto-Pinto model (Steel02 in OpenSees) was used to

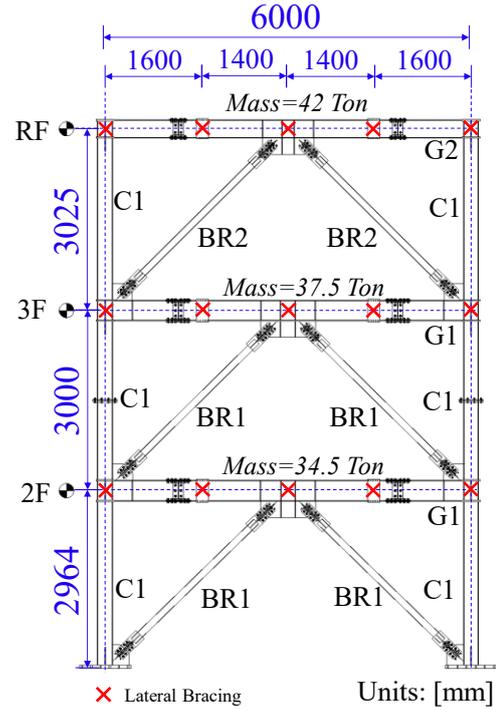


Fig. 1 Specimen

define the material properties, based on assumed yield strengths. A 0.1% in-plane initial imperfection of the brace length was applied to each brace. The panel zones were modeled following the formulation proposed by Gupta and Krawinkler [5] by seven rigid forced-based elements connected by three hinges and a rotational spring to represent shear response, although yielding of the panel zone is not expected. The gusset plates were modeled using the approach proposed by Hsiao et al. [6], with a rotational spring to simulate its out-of-plane rotational stiffness. Nodal masses were applied directly to the columns in the horizontal degree of freedom. Low-cycle fatigue on braces and beam degradation was not modelled.

5. Simulation Results

A nonlinear pushover analysis was conducted using nodal displacement control and the Newton-Raphson algorithm following the lateral force distribution patten computed as: 2.56F for the roof floor (RF), 1.47F for

Table 1 Frame members

ID	Section	Designation	Expected σ_y [MPa]	D/t KL/r		Rank ^[7]	Ductility ^[8]
C1	□HSS-250×250×9	BCR295	376	-	-	FA	MD
G1	H-350×175×7×11	SS400	330	-	-	FA	MD
G2	H-300×150×6.5×9			-	-	FA	MD
BR1	ØHSS 114.3×4.5	STK400	380	33	95	FC	HD
BR2	ØRHSS 114.3×3.5			25	94	FC	MD

the third floor (3F), and 1.0F for the second floor (2F). From the pushover curve shown in Fig 2, it can be observed that the ultimate lateral strength after brace buckling coincides with the estimated values of the expected yielding strength. In addition, the failure mechanism of the specimen is verified, and it is observed that the largest deformations are expected to concentrate on the first two stories. Additionally, the story-drift ratio at the story drift ratio at which the braces start to buckle in the first story is 0.0019 rad. In addition, to anticipate the performance of the specimen, a time history analysis was conducted using the Kobe JR Takatori NS (1995) ground motion scaled to 25%, 50%, 75% and with three repetitions each at 100%, and a 60-s rest between adjacent motions to eliminate free vibration. These accelerations were adjusted to reflect the specimen's weight and the capacities of the shake table facilities. Damping was simulated using the Rayleigh model, with a critical damping ratio of 0.02, applied to the first and third natural periods. Numerical integration was performed using Newmark's method ($\beta=0.25, \gamma=0.50$), with a time step of 0.01 seconds.

Fig. 3 shows the shear force versus story drift-ratio at each story, where the maximum story drift-ratio of 0.034 rad can be identified. In all three cases, the hysteresis loops evolve progressively from 100%-1 to 100%-3, indicating cumulative damage and stiffness degradation. The responses of the braces, compared with the computed properties, are shown in Fig. 5 for each story.

The maximum floor acceleration, shown in Fig. 4(a), is essential for designing the inertia transfer system, which connects the mass source-precast concrete slabs to the specimen through collectors directly connected to the columns. The maximum acceleration exceeded 1g during the 100% excitations.

Fig. 5 (b) shows the distribution of the maximum story

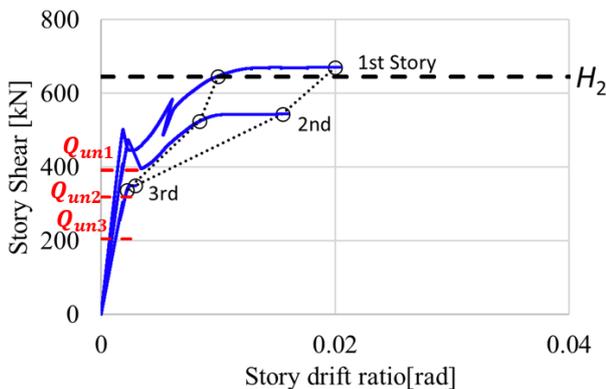


Fig. 2 PushOver Analysis assuming $R_y F_y$

drift ratio and illustrates the progressive stiffness degradation.

6. Summary and conclusion

This study presents the test plan and pre-test numerical simulation of a full-scale three-story steel Chevron Braced Moment-Resisting Frame designed for shake-table testing. The specimen was developed within a Japanese design framework while

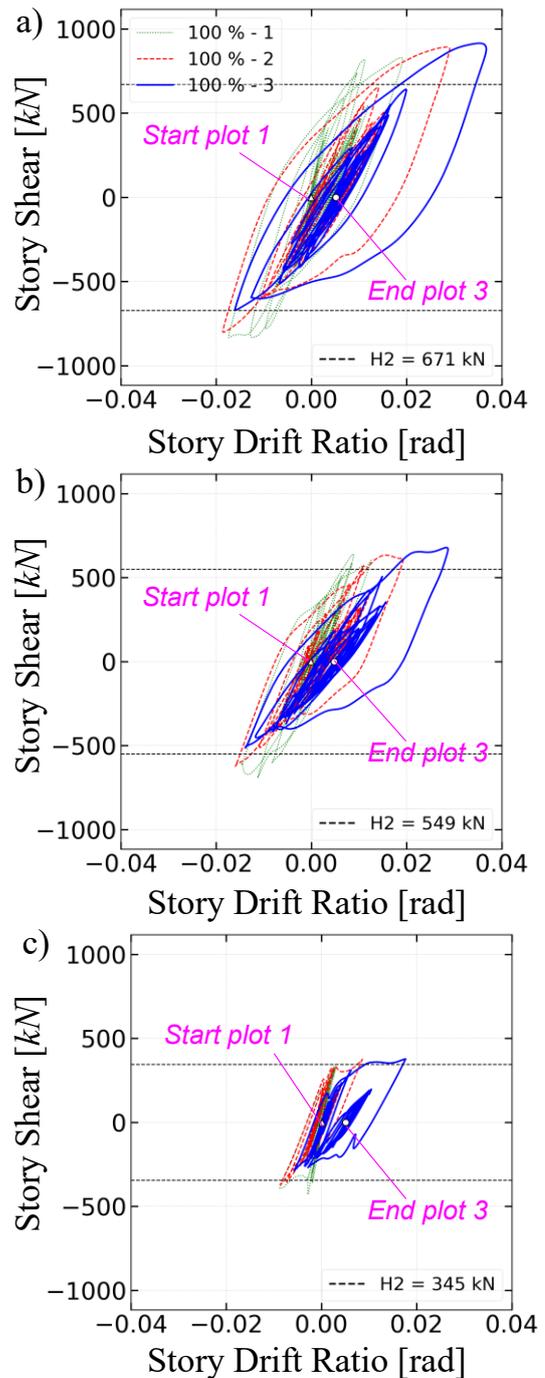


Fig. 3 Shear Force versus Story drift ratio:
(a) First; (b) Second (c); Third story

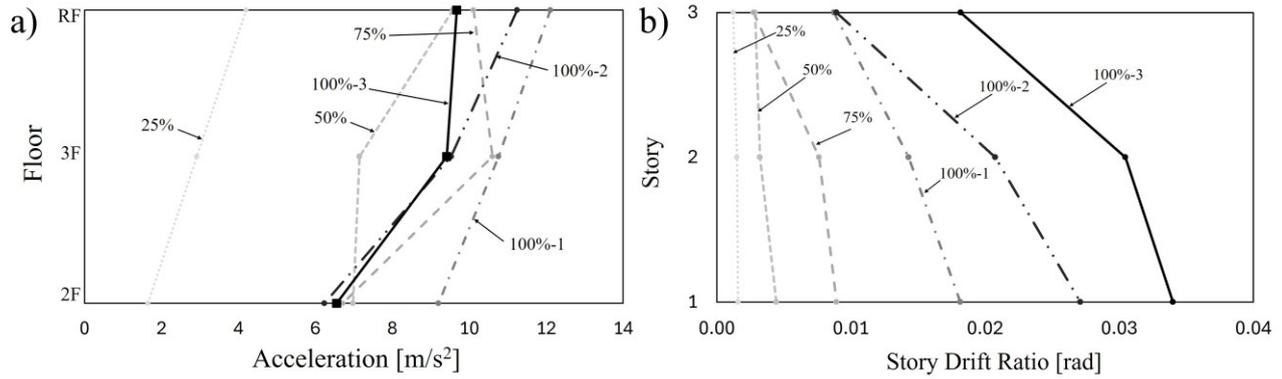


Fig.4 Simulation maximum responses:

(a) Maximum floor acceleration; (b) Maximum story-drift ratio

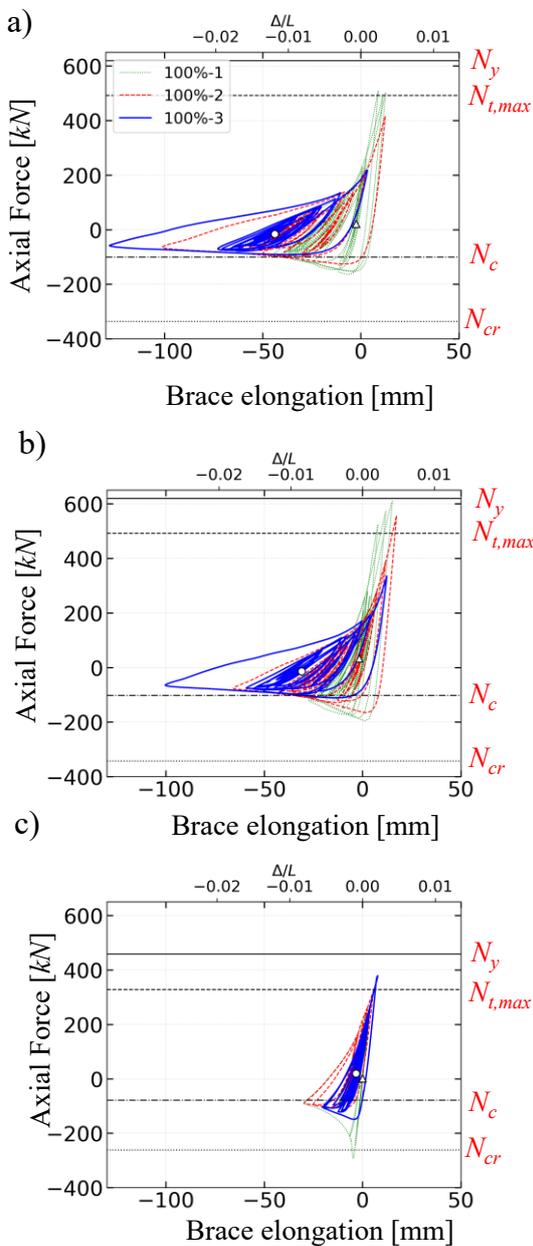


Fig. 5 Braces response for:

(a) First; (b) Second; (c) Third story

benchmarking ductility limits, with capacity-design checks applied to members, connections, and splices to ensure controlled inelastic behavior. A nonlinear OpenSees model incorporating geometric nonlinearity and idealized member and connection behavior was employed to capture global response trends, including strength, stiffness degradation, and load redistribution. The model served as a pre-test planning tool to corroborate expected mechanisms and to anticipate the specimen's response. Analytical results established baseline expectations for story drift demands, brace axial hysteresis, vertical deflections, and stiffness degradation indicators such as period elongation. Degradation and fatigue were not modeled; post-test calibration is needed to capture cycle-dependent and fracture-related behavior.

References

- 1) R. Mamani, "An Experiment on Steel Chevron Braced Moment Resisting Frames," M.Eng. thesis, Division of Architectural and Structural Design, Graduate School of Engineering, Hokkaido University, Master Thesis No. M632, Aug. 2025.
- 2) C.-C. Chou, H.-Z. Lin, A. Córdova, J.-M. Chen, D. Y.-H. Chou, S.-H. Chao, S.-H. Chao, G. Tsampras, C.-M. Uang, H.-Y. Chung, C.-H. Loh, and H.-T. Hu, "Earthquake simulator testing of a three-story steel building for evaluating built-up box column performance and effect of sliding slab," *Earthq Engng Struct Dyn*, vol. 53, pp. 2637–2655, 2024, doi: 10.1002/eqe.4130.
- 3) McKenna F: Object oriented finite element programming frameworks for analysis, algorithms and parallel computing, PhD thesis, University of California, Berkeley, California, 1997
- 4) Karamanci E et al: Computational approach for collapse assessment of concentrically braced frames in seismic regions, *J. of Struct. Eng.*, 140(8), A4014019, 2014.
- 5) Gupta, A and Krawinkler, H. (1999). Seismic Demands for Performance Evaluation of Steel Moment Resisting Frame Structures. John A Blume Earthquake Engineering Center Technical Report 132. Stanford Digital Repository. Available at: <http://purl.stanford.edu/fm826wn5553>
- 6) P. C. Hsiao, D. E. Lehman, and C. W. Roeder, "Improved analytical model for special concentrically braced frames," *J. Constr. Steel Res.*, 2012, doi: 10.1016/j.jcsr.2012.01.010.
- 7) MLIT, Commentary on Structural Regulations of the Building Design Standard Law of Japan. (Ministry of Land, Infrastructure, Transport and Tourism), 2020.
- 8) AISC, 341 "Seismic Provisions for Structural Steel Buildings dated and all previous versions Approved by the Committee on Specifications," 2022.