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STRUCTURAL MODELS FOR CENTRIFUGE TESTING OF LIQUEFACTION-RELATED BUILDING DAMAGE

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Abstract

This paper describes the design of more realistic structural models for centrifuge experiments that are used to characterize the response of damageable structures on liquefiable soils. These experiments are needed because seismic design of structures commonly ignores soil-structure interaction, while techniques for evaluation and mitigation of liquefaction generally fail to consider potential damage to the structure and inertial interaction effects. Here, a three-story steel moment frame was designed based on code requirements and subsequently simplified and scaled for centrifuge experiments. Component tests were performed at 1g and 70g of centrifugal acceleration to characterize the nonlinear properties of beam-column subassemblies. The model structure was then placed on a layered soil deposit, including a liquefiable layer, in centrifuge. Results in terms of moment-rotation were obtained experimentally from component tests as well as the centrifuge experiment, which were compared with OpenSees simulations performed during design. As expected, the behavior of the components at 1g and 70g were similar. The structure did not experience any damage and remained essentially elastic when founded on liquefiable ground, because energy was dissipated through settlement and tilt; inelastic structural response, however, is possible when if liquefaction is mitigated. The response was also found to agree well with the OpenSees model.

Keywords: Soil Structure Interaction; Steel Frames; Centrifuge; Liquefaction; Pushover.

1. Introduction

The response of a structure in an earthquake is strongly influenced by its interaction with foundation and the surrounding soil. Depending on earthquake characteristics and soil and structure properties, soil-structure interaction (SSI) effects can be detrimental or beneficial for structural performance. Yet, design codes provide guidance for engineers based on simplified procedures, which do not capture factors such as permanent deformations due to soil liquefaction combined with ground shaking.

Potential damage to a shallow-founded structure on liquefiable soils is generally attributed to excess pore pressure buildup during shaking that results in significant permanent building settlement and tilt. The state of practice for evaluating building settlements on liquefiable ground rely on empirical procedures that assume free-field conditions, not taking into account the influence of the building. These procedures have been shown to be unreliable [1], [2]. Further, our understanding of the influence of the combination of ground shaking and settlement due to liquefaction on building performance is limited, particularly when liquefaction mitigation techniques are employed. Previous case histories have shown the important influence of building properties, such as foundation width and structure's height/width ratio on permanent settlement. Yet, a thorough evaluation of soil and structural response from case histories is often hindered by lack of adequate instrumentation and recordings, and few experiments have considered potentially damageable (inelastic) structures on liquefiable ground. Hence, experimental research is needed to model the soil-structure system and evaluate the effects of key parameters systematically, which can be used in validating advanced numerical tools used in design.

In this study, centrifuge experiments were carried out in order to quantify the nonlinear behavior of steel structures on liquefiable soils. A three-story steel building was designed according to modern U.S. seismic design codes for high seismic regions (i.e., following ASCE [3] and AISC [4] requirements), which was subsequently simplified and scaled for centrifuge testing. This paper focuses on the design of the structure, the characterization of the response in the component tests, and the validation of this response in centrifuge. The paper describes first the static component tests at 1g and 70g of centrifugal acceleration conducted to obtain the elastic and inelastic mechanical properties of the beam-column subassemblies that comprise the model frame structure. In addition, the overall performance of the building on liquefiable soils was evaluated by a dynamic soil-structure test conducted at 70g of centrifugal acceleration with multiple earthquake motion records. Detailed examination of the model structure was needed because beam-column nonlinearity using replaceable fuses has not been previously employed in centrifuge tests for realistic multi-degree-of-freedom structures resting on liquefiable soils. The state of knowledge and practice for analyzing the performance of damageable structures on liquefiable soils with and without mitigation is still developing.

2. Background

The behavior of steel frame structures is controlled by its beam-column subassemblies and their connections. During the Northridge 1994 earthquake many beam-column connections in steel moment-resisting frames were damaged. Since this event, AISC [4] has required those connections to be capable of developing at least 0.4 radians of interstory drift in order to provide the ductility necessary to dissipate seismic energy. In addition, modern frame design aims to avoid transmission of large moments from beams to columns. Therefore, much research has been carried out on the behavior of modern code-designed beam-column subassemblies. For example, Sato et al. [5] tested three full-scale moment connections for special moment frames. Similarly, cyclic tests of reduced beam sections connected to deep columns were performed by [6]. Lignos et al. [7] used an experimental database to recommend cyclic moment-rotation relationship for modeling plastic hinge regions in steel beams. However, much less work has been done on the behavior of steel beam-column subassemblies for structures with a flexible base and subject to potentially severe differential foundation settlements.

Centrifuge tests have been used regularly for examining SSI problems [8]–[11]. In the past, the behavior of structures founded on shallow foundations and liquefiable soil deposits has been studied in terms of settlement and excess pore pressures in the underlying soil. In most cases, the structure was represented as a rigid block or a bearing pressure. For example, Liu et al. [12] investigated the mechanisms of liquefaction-induced settlement under a circular rigid footing placed on a medium dense, saturated sand deposit. Yoshimi et al. [13] conducted 1g shake table tests on rigid structures made of steel or concrete placed on saturated sand, observing a reduction of structure settlement as the width of the footing increased.

Recently, centrifuge tests have been performed in order to identify settlement mechanisms of linear-elastic, single-degree-of-freedom (SDOF) buildings with rigid mat foundations on liquefiable soils. Dashti et al. [1] showed how the characteristics of the structure, soil profile, and the ground motions influenced excess pore pressure generation and settlements in the underlying soil. Meanwhile, Hayden et al. [14] investigated the physical restraint imposed by an adjacent foundation on neighboring structures on liquefiable ground, examining the reduction in foundation settlement. Finally, Trombetta et al. [15] performed centrifuge experiments on steel frame structures with: (1) a SDOF model on spread footings; and (2) a three-degree-of-freedom (3DOF) model on a deep basement. Both models were tested on dry Nevada sand (no liquefaction) to examine the seismic response of inelastic structural models on dry soils and their interaction with each other through soil. The response of inelastic, multi-degree-of-freedom (MDOF) structures has not been studied experimentally on liquefiable ground, which hinders our understanding of building performance and damage potential on softened soils and the effectiveness of mitigation strategies. In this paper, performance of a code-designed steel frame building tested on liquefiable soils in centrifuge is discussed.

3. Design of Building for Centrifuge Testing

3.1 Design overview

The design of the structure was constrained by experimental and and realistic conditions related to both prototype (the full-scale building we are trying to represent in the experiment) and model scale. In designing the overall dimensions of the structures, facility constraints and centrifuge scaling laws needed to be considered. The large aluminum flexible-shear-beam container at the University of Colorado Boulder with inside dimensions W x L x H= 37.6 x 96.8 x 30.4 cm was employed in this study. Our goal was to model two separate structures simultaneously in each test. These buildings needed to be far enough apart from each other and the borders of the container to avoid interference and boundary effects. These calculations permitted modeling up to two separate structures with base dimensions of foundation 13.6 x 13.6 cm in model scale. A centrifuge scale of N = 70 was selected, in order to achieve a realistic prototype foundation dimension of 9.5 x 9.5 m, while limiting the overall weight applied to the shaking table under increased gravity.

In addition to this consideration, other constraints were also taken into account in design. First, our objective was to design a fully code-conforming structure designed based on ASCE [3] and AISC [4] requirements. However, due to the container size limitations described above and constructability of steel sections, the structure was limited to a 1-bay moment resisting frame. In addition, a height/width ratio value between 1.5 and 2.0 and a foundation contact pressure of approximately 70 to 80 kPa was targeted to amplify soil-structure interaction effects, settlements, and tilting tendencies. The foundation embedment needed to be calculated to ensure overall stability (no bearing capacity failure) even after liquefaction, in order to limit settlements to the small to medium range. Finally, the design was constrained by requirements related to instrumentation application; for example, the reduced section “fuse” needed to be at least 7.6 mm wide to provide enough space for placing of strain gauges.

3.2 Target 3-story prototype building

A three-story, one-bay, special steel moment resisting frame building was designed based on the ASCE 7-10 [3] requirements and the overall prototype dimensions determined in Section 3.1. The structure was assumed to be located at a high seismic California site with soil class D, even though the structure was later placed on a liquefiable layer soil deposit in centrifuge (soil class F). The equivalent lateral force procedure (ASCE 7-10 Section 12.8) was used to obtain sections for beams and columns based on drifts and strength requirements. A live load of 2.4 kPa and a dead load of 26 kPa were used in design of the structure.

The designed building has plan dimensions of 7.1 x 7.1 m, with 1 bay and a height of 4.5 m for the 1st story and 3.9 m for the 2nd and 3rd stories. Steel W24x131 sections were chosen for beams and columns to satisfy design requirements. As is typical for these kinds of buildings, drift governed the section selection, and the design drift of 1.8% was less than 2% specified in ASCE 7-10. Strong-column-weak-beam and other design rules were satisfied. Since drift limits controlled, section sizes were selected to be larger than those required by strength [16] alone. The contact pressure of the structure was 75 kPa.

3.3 Simplified 3-story building for centrifuge testing

The prototype structure was simplified and scaled as two identical steel frames connected by concentrated masses and steel lateral plates to represent a three-degree-of-freedom (3DOF) system. Due to the complexities in scaling relationships, we were not able to represent all properties of the structure precisely in the scaled model. Beam-column moments of inertia were of primary interest, because element response was expected to be primarily flexural. Therefore, member area was not scaled exactly due to limitations in available material sections and restrictions on fuse sizes for strain gauge application. In order to represent the potential nonlinearity of the structure, sections at the ends of beams and columns were reduced in “fuses”. The fuse design ensured that flexural yield strength in beams was reached and inelastic deformations were kept out of the columns in the model structure, thereby satisfying strong-column-weak-beam criteria. In the model building, inelasticity is concentrated at these fuse locations. A solid rectangular cross section of 7.6 x 4.75 mm (scaled model) was determined for the reduced section for beams and columns by scaling the moment of inertia of the designed beam in prototype scale by N^4. The fuse size was also governed by the available sections provided by the manufacturer and the necessary size needed to place strain gauges. Based on manufacturer specifications of steel strength, the following mechanical properties (in model scale) for simplified building were used in design: yield moment capacity of 10.7 N.m, and yield stress of 372 MPa. The rest of the beam and column sections were somewhat larger (see Table 1) for ease of constructability and to prevent nonlinearity outside the fuses. Table 1 summarizes the main geometric properties of beam, column, and fuses in model scale. A 13.6 x 13.6 x 1.5 cm foundation was constructed for the scaled building to represent a rigid mat foundation. The foundation was constructed using aluminum for practicality.

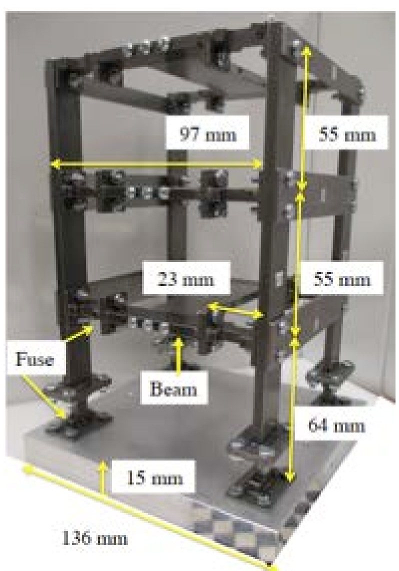


Fig. 1 – Schematic illustration of the scaled three-story steel frame model building used in centrifuge. All dimensions are in model scale.

4. Structural Analysis Model

The simplified structure used in the centrifuge was modeled in two-dimensions using OpenSees. Elastic beam-column elements [17] were used to model beams and columns of the simplified building, and fiber steel sections were used for modeling the fuses based on nonlinear beam-column elements [17] with a uniaxial steel material, which combines linear kinematic and isotropic hardening [17]. Column and beam self-weights as well as the tributary weight due to the slab and partitions (represented by the lumped mass in the simplified building) were accounted for by applying a distributed load to the beams. Mass was aggregated to the nodes by their tributary area in both plan and elevation. In other words, half of the total mass of the building was distributed to the various nodes in the 2D model.

Modal analyses to identify the fixed-base natural frequencies of the simplified 3-story structure (Section 5.1) were carried out on the simplified model with fully restrained nodes at the bottom of the building. In addition, a nonlinear pushover analysis under the same boundary conditions was performed as a blind prediction of building behavior (section 5.5).

Table 1 – Geometric properties of beams, columns, and fuses of a scaled model building in centrifuge (properties provided in prototype scale).

	Beam & Column	**Fuse**
Shape	Solid rectangular sections	
Moment of inertia (cm^4)	343100	164600
Area (cm^2)	3700	1780

5. Experimental Investigations of Fabricated Structure

5.1 Free-vibration impact tests

The purpose of these tests was to characterize damping and the natural frequencies of the building, which can be compared with OpenSees estimates under fixed-based conditions. Since the model structure can be idealized as a 3DOF system with lumped masses at each floor connected by massless columns, the test quantifies three modal frequencies.

Three impulse impacts were applied by a rubber hammer, as shown in Fig. 2a, in the shaking direction. The acceleration time history was recorded at each floor by accelerometers. The transfer functions (TF) of accelerations were obtained by dividing the Fourier amplitude of acceleration on each story by that of the base plate as a function of frequency, as shown in Fig. 2b. The peaks of the response correspond to the resonant frequencies of the fixed-base structure; in other words, these were interpreted as natural frequencies of the structure. Small differences between the impact tests results in terms of frequencies and damping were observed. Table 2 provides the natural period(s) of vibration from the impact test results and numerical model in OpenSees in prototype scale. According to these results, there is good agreement between the model and the impact tests. The small differences were attributed to the mass of instruments in the physical model. A damping ratio of approximately 5.3% was computed using the logarithmic decrement method [18].

5.2 Beam-column component tests

Component tests were used to characterize the nonlinear behavior of the fuse sections. Monotonic and cyclic tests under 1g and 70g of centrifugal acceleration were performed in the medium (15g-ton) centrifuge at University of Colorado Boulder. The set up consists of a hypothetical beam and column section, as shown in Fig. 3b. where columns were attached by pinned connections to the fixities, to allow the column to rotate freely. The fuses (illustrated in Fig. 3c) were designed to be replaceable and were attached at beam column connection.

This kind of test is not commonly performed in centrifuge and, therefore, some challenges were encountered during the test. One of these was the application of a point load at the end of the beam. The load cell was connected to the beam with a pin connected through a hole where a horizontal gap was necessary to provide enough space to allow the beam to rotate freely under the applied load without restricting its vertical movement. The test set up needed to be capable of characterizing the response of the fuses up to and exceeding the 0.04 radians expected in modern steel connection design.

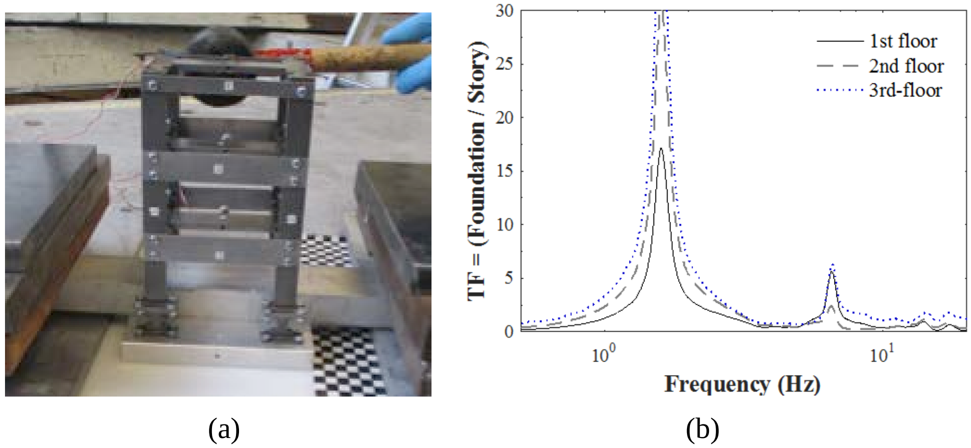


Fig. 2 – (a) Impact test setup; and (b) transfer functions of acceleration at each floor with respect to the base of the fixed-base structure recorded during one the impact test.

Table 2 – Modal frequencies (Hz) [Periods (s)] for the 3-story building obtained numerically and experimentally.

	Mode 1	**Mode 2**	**Mode 3**
Impact Test	1.7 Hz [0.58 s]	6.7 Hz [0.15 s]	16.7 Hz [0.06 s]
OpenSees Simulation	2.0 Hz [0.50 s]	7.7 Hz [0.13 s]	16.7 Hz [0.06 s]

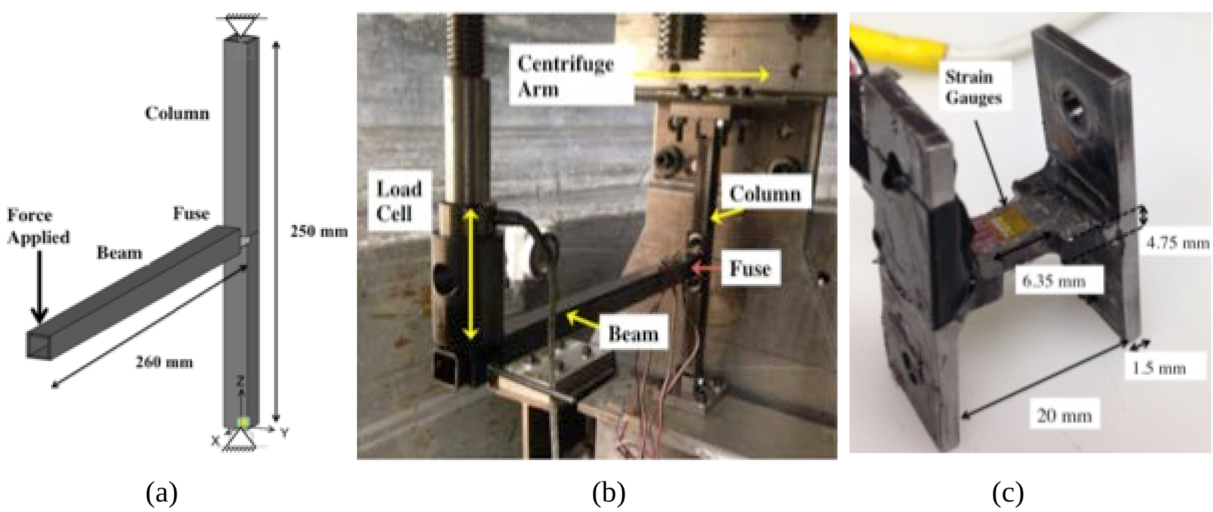


Fig. 3 – Fuse component testing: (a) diagram of test; (b) test setup inside centrifuge; and (c) fuse detail.

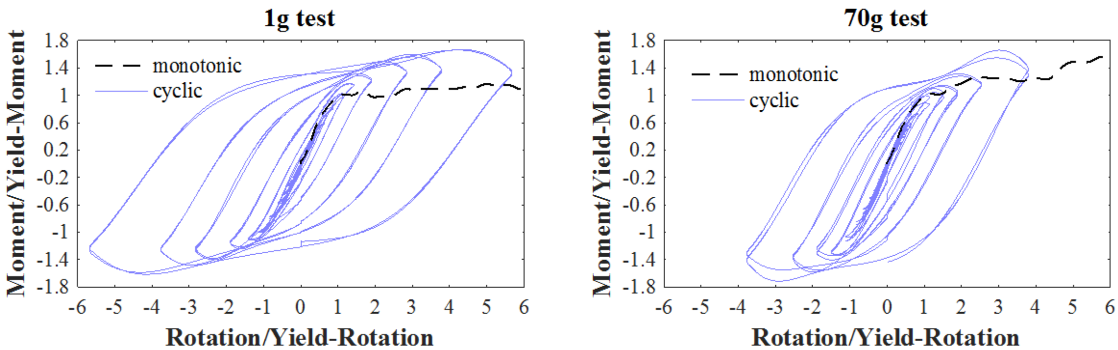
Eight tests were conducted by applying a displacement-controlled command at the end of the beam including two sets of two monotonic and two cyclic both at 1g and 70g. To perform the monotonic tests, a vertical displacement was applied to the end of the beam. The cyclic test followed an established loading protocol for cyclic testing of steel components [19]. Both types of tests were continued to as large of a displacement as possible; the end of the test was typically governed by the displacement reaching the capacity of the test setup.

Strain gauges were placed on both sides on the fuses in Fig. 3c to measure their stress-strain behavior. They were set up in a half-bridge configuration at top and bottom of each fuse. Rotation was computed based on the imposed displacement and total length of the beam-fuse system. The moment on the fuse was calculated based on the load cell data and total length of the beam-fuse system.

5.3 Beam-column component test results

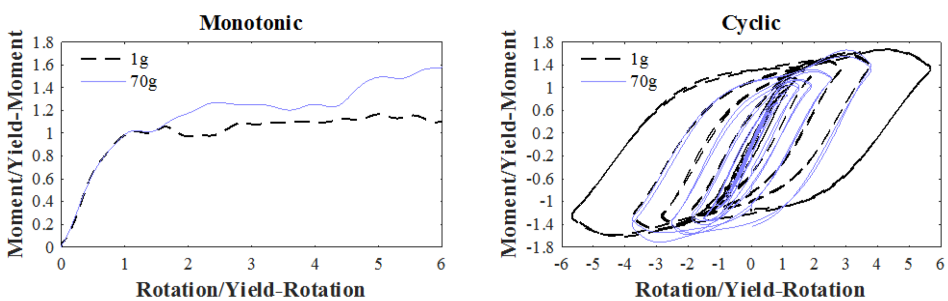
Eight component tests were performed in order to characterize the fuse section and its behavior under monotonic and cyclic loading. The mean (μ) and standard deviation (σ) values were estimated for yield moment and rotation as follows: 1) yield moment μ = 16.9 N.m and σ = 2.78 N.m; and 2) yield rotation μ = 0.032 rad and σ = 0.0037 rad. These results were used to compute yield strength of the steel as 588 MPa, which is approximately 58% larger that the originally expected design value of 372 MPa provided by the manufacturer. Differences in steel strength compared to manufacturer-reported values are commonly observed. However, this difference led to a stronger achieved fuse behavior than anticipated or desired.

Fig. 4 and Fig. 5 show the moment-rotation plots obtained from the component tests, normalized by yield moment and yield rotation. No major difference was observed between component tests at 1 g and 70 g of centrifugal acceleration because of the low mass of elements tested and the expected similitude in stress and strain [20], suggesting that the moment-rotation behavior of structural elements is independent of centrifugal acceleration applied. Also, a similar initial stiffness was observed in all specimens tested. However, some differences could be noticed in terms of yield strength, and, particularly, monotonic response showed less strength than cyclic. This is consistent with the expected cyclic strain hardening behavior of steel [21].



(a) (b)

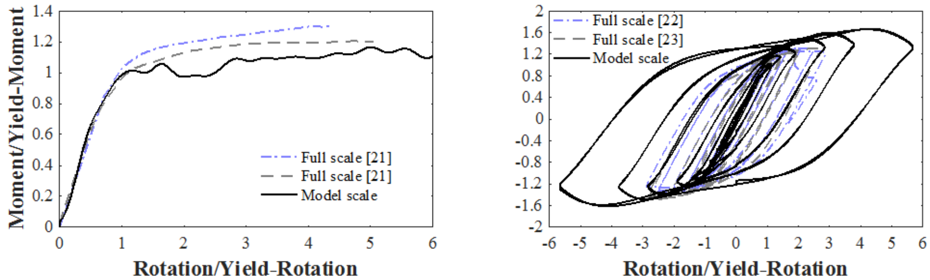
Fig. 4 – Moment-rotation relationship of beam-column subassemblies from monotonic and cyclic tests at: (a) 1g; and (b) 70g of centrifugal acceleration.



(a) (b)

Fig. 5 – Moment-rotation relationship of beam-column subassemblies from: (a) monotonic tests; and (b) cyclic tests at 1g and 70g of centrifugal acceleration.

To evaluate the behavior of the steel beam-column subassemblies used in these scaled, simplified centrifuge models with those of realistic, full-scale, steel structures, a comparison of monotonic and cyclic tests was made between results in this study and full-scale tests presented by [21]–[23]. These results were normalized with respect to yield moment and yield rotation to compare initial stiffness and post-yield behavior at different scales. One of the full-scale tests have beams with so-called "reduced beam sections" (RBS) [23]. This section is a real design approach that is similar to the fuse condition in our simplified model. The rest of the full scale specimens do not have RBS, but have section sizes and characteristics similar to the prototype structure design. In Fig. 6, the monotonic and cyclic response of the steel beam-column subassemblies show similar shapes in terms of stiffness and yield strength between the full-scale and simplified model test results. This comparison provides confidence that the post-yield response and cyclic behavior in the simplified beam-column is similar enough to the full-scale situation we aim to represent. The main difference was observed in the cyclic response where our component test results (labeled "cyclic-1g" indicating cyclic tests at 1g) showed larger loops than the full-scale results. This is because more cycles and greater deformations were applied during this test compared with the full-scale tests presented, i.e. our tests were continued to larger deformations.



(a) (b)

Fig. 6 – Moment-rotation relationships of beam-columns comparing simplified model subassemblies with representative full-scale tests under: (a) monotonic; and (b) cyclic loading.

5.4 Centrifuge experiments of the soil-structure system

A dynamic centrifuge test was subsequently carried out using the large (400g-ton) centrifuge at the University of Colorado Boulder to gain a better understanding of the behavior of more realistic MDOF steel structures with rigid mat foundations on liquefiable layered soils. As mentioned previously, the properties of structure and soil layers were selected to amplify soil-structure interaction (SSI) effects on softened soil deposits, and the instrumentation was designed to evaluate the overall performance of the building in terms of modal periods, drifts, foundation settlement, tilt, and fuse moment-rotation behavior.

The liquefiable soil model was constructed in a new flexible-shear-beam (FSB) container made of aluminum and rubber to simulate the response of soil in its softened state and reduce boundary effects. A dense layer of Ottawa sand (c_u =1.56, e_{min} =0.53, e_{max}= 0.81, G_s = 2.65) was pluviated with a relative density (D_r) of approximately 90% at the bottom of the container, extending up to 12 m in prototype scale. Then, 4 m of loose Ottawa sand was pluviated with a $D_r \approx 40\%$ as the liquefiable material. Lastly, 2 m of Monterrey sand (e_{max}= 0.84, G_s = 2.65) with $D_r \approx 90\%$ was pluviated up to the surface. The structure was embedded 1 m in the Monterrey sand. The phreatic level was kept slightly above the ground surface to ensure complete saturation of the model even after spin up. Initially, the model was flushed with CO_2, then, it was kept under vacuum, while the viscous fluid was slowly added from bottom up. A hydroxypropyl methylcellullose solution with a viscosity of approximately 64 cSt (approximately 70 times water) was used as a pore fluid to satisfy the scaling laws.

Fig. 7 presents an elevation view of the instrumentation layout of one of the centrifuge experiments performed in this study, which is discussed in this paper. Two identical building structures were placed in the container, one with no mitigation (A) and one with densified ground (A-DS). Only the results from Structure A (with no mitigation) are discussed here, to examine how the structure's response in centrifuge compared with design. Ninety-one sensors were used during this test including accelerometers, pore pressure transducers (PPTs), LVDTs, and strain gauges. For this paper, mainly accelerometer and strain gauge recordings are presented to evaluate the flexural drift and moment, respectively, of fuse sections

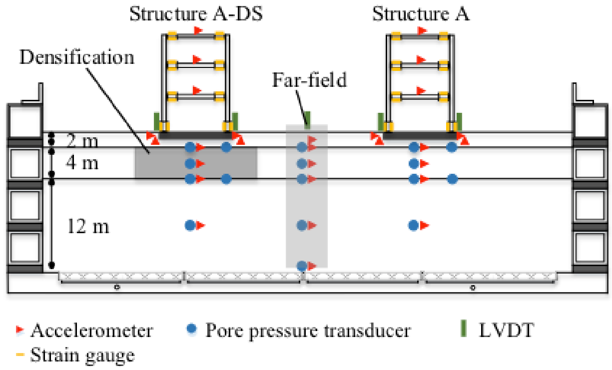


Fig. 7 – Instrumentation layout of the centrifuge model test.

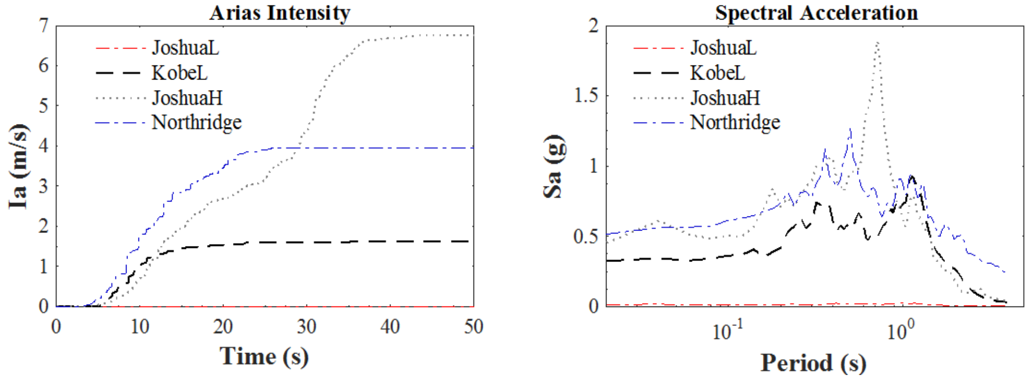
A series of earthquake motions was applied at base of the container in flight (Table 3), leaving enough time between the application of motions to permit dissipation of excess pore pressures. The selected motions were scaled versions of the horizontal component of different earthquake recordings with different properties. The first motion (JoshuaL) was applied for characterizing soil small-strain shear wave velocity; consequently, the initial fundamental period of vibration of the soil profile was computed (T_{so} = 0.5 sec). In addition, this motion allowed us to ensure all sensors were working well. The KobeL motion liquefied the soil in the far-field but not under the building, then JoshuaH and Northridge motions were applied, causing significant settlement and tilt of the structure as well as liquefaction under the corners of the building. Fig. 8 shows the Arias intensity time histories and acceleration response spectra (5% damped) of the base motions as recorded in centrifuge.

Table 3 – Ground motion properties as recorded at the base of the container in centrifuge.

Ground Motion ID	**Event**	**Station**	**PGA (g)**	**Sign. Duration, $D_{5\text{-}95}$ (s)**	**Mean Period, T_m (s)**	**Arias Intensity, I_a (m/s)**
JoshuaL	1992 Landers	Joshua Tree	0.01	32	0.9	0.0041
KobeL	1995 Kobe	Takatori	0.33	14.3	0.9	1.6
JoshuaH	1992 Landers	Joshua Tree	0.43	27	0.7	6.8
Northridge	1994 Northridge	Newhall-WPC	0.51	16.4	1.1	3.9

5.5 Structural response in the centrifuge test

We quantified the structural response in the dynamic centrifuge test based on the moment-rotation response of the columns. In the centrifuge test, the rotation of the column fuses was approximated as the roof flexural drift ratio. This flexural drift ratio was computed as the difference between the total and rocking drift ratios as described by [24]. The total roof drift was obtained by double integrating and subtracting accelerometer recordings at the roof and foundation. This drift was then normalized by the height of the building to obtain the total drift ratio. The vertical accelerometers at the foundation edges provided a measure of foundation rocking and the resulting roof drift ratio due to a rigid body rotation, which was removed from the total drift calculations to compute the flexural component of roof drift. Moment was computed directly from the strain gauges on the column fuses. In this part of the study, we compare the centrifuge test results to results from pushover test of the OpenSees model described in Section 4. In the pushover analysis performed in OpenSees, we applied an inverted triangular load and recorded the moment in the column fuses and the roof (flexural) drift. These quantities can be directly compared to the results obtained during the centrifuge experiment.



(a) (b)

Fig. 8 – Properties of base earthquake motions as recorded in centrifuge, reporting: (a) Arias Intensity time histories; and (b) 5%-damped acceleration response spectra.

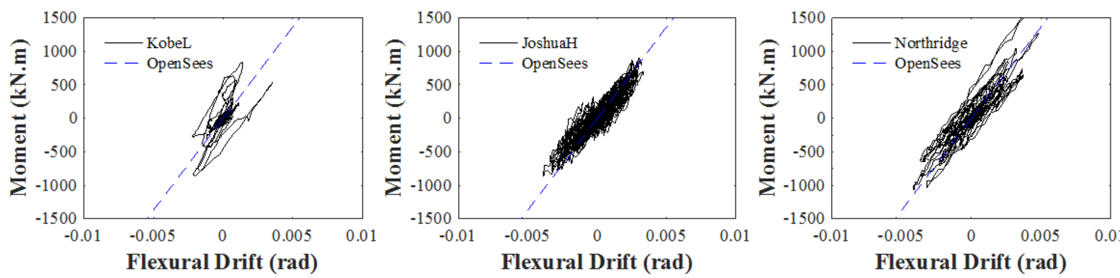


Fig. 9 – Moment-rotation relationship from OpenSees and centrifuge test results, showing results for each ground motion individually.

A comparison between the OpenSees analysis results and the response of fuses as recorded in centrifuge is provided in Fig. 9. These plots demonstrate that the model captured well the expected stiffness and response of the structure in the elastic range. In addition, the results show that the yield strength of the structure was not reached during any of the motions applied in centrifuge. The maximum base shear experienced by the structure during the centrifuge test was 1258 kN, which was greater than the base shear design value (289 kN), but lower than the strength of the structure. Inelastic behavior of code-designed structures is possible for shaking levels exceeding an intensity 1/R of spectrum design, where R = 8 is the design seismic response modification coefficient. However, many steel moment frames behave elastically up to 1/3 of the design spectrum or even higher due to design requirements other than strength that govern section size selected [16]. In addition, in the simplified model structure, there was significant overstrength in the steel used in the fabricated fuses compared to the manufacturer-reported values used in design. Moreover, the overstrength was amplified by use of a one-bay structure because of space limitations in centrifuge. Further, the structure on liquefiable ground experienced excessive permanent settlement and rocking, which reduced the demand transferred to the superstructure and, hence, flexural deformations experienced at the fuses.

6. Conclusions

Centrifuge experiments were performed to examine the behavior of 3DOF steel frame structures with rigid mat foundation on liquefiable layered soils. Monotonic and cyclic tests under 1g and 70 g centrifugal acceleration were performed to characterize the nonlinear properties of beam-column subassemblies. Additionally, a centrifuge test at 70g of centrifugal acceleration was carried out to assess the overall performance of soil-structure system and validate the frame response.

The component tests revealed that no major differences were observed in the mechanical behavior of the steel fuse at 1g and 70g. In addition, these tests showed a good agreement in terms of post-yield hardening and cyclic behavior between our simplified beam-column and full-scale tests from the literature. The component tests were also used to quantify the expected steel strength for model development. In the centrifuge test, flexural drift and force demand was limited, since the seismic energy was mostly damped by liquefaction of the underlying soil. However, these results showed that the OpenSees model provides good estimates of stiffness and flexural response of the structure in centrifuge tests. Even though, challenges were identified regarding the design of an inelastic structure in centrifuge. Specifically, space constraints in centrifuge led to the design of a 1-bay structure that may be unrealistic, and steel properties may be misrepresented by the manufacturers, both leading to overstrength in the structure and difficulty in capturing deterioration of strength and stiffness in fuses under earthquake loading.

This research helped evaluate the behavior of buildings designed based on code requirements on liquefiable soils for centrifuge modeling. Based on these observations and limitations, to evaluate the performance of inelastic buildings that can be damaged on liquefiable ground with and without mitigation, weaker fuses are currently being designed by the authors for the same structure. Future tests will also examine the impact of different mitigation techniques on the performance and damage potential of structures.

7. Acknowledgments

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