Physical modelling of reinforced concrete at a 1:40 scale using additively manufactured reinforcement cages

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Abstract
Global level assumptions of numerical models have received relatively less attention, but have been indicated to be a major source of error in numerical modeling of Reinforced Concrete (RC) structures. In parallel, it has been stated that a statistical approach involving many virgin specimens and ground motions is necessary for model validation. Such an approach would require very small-scale testing. Then, the reinforcement fabrication becomes a major issue. This paper proposes using additive manufacturing to fabricate the reinforcement cage. It presents the results from cyclic tests on 1:40 RC cantilever members. The cages were manufactured using an SLM 3D printer able to print rebars with submillimeter diameters. Different longitudinal and transverse reinforcement configurations were tested. A numerical model using existing OpenSees elements was built and its parameters were calibrated against material level small-scale tests. It captured the cyclic response of the RC members with a reasonable accuracy. The cyclic behavior of the RC members resembles the behavior of full-scale RC members indicating that such small-scale specimens can be used for the statistical validation of the global level assumptions of numerical models.

Keywords
additive manufacturing, centrifuge testing, experimental earthquake engineering, physical modeling, reinforced concrete

1 INTRODUCTION

Small-scale testing of RC structures was the rule some 40 years ago, when large shake tables did not exist. The interested reader is referred to Harris and Sabnis for a list of references discussing physically modeling concrete at scales on the order of 1/5–1/8. As concrete properties are scale-dependent (among many others), small scale models (smaller than 1:5) were abandoned when larger shake tables were constructed. Nowadays, there are two reasons to test RC models at a very small scale, such as 1:40 (or smaller).
The first reason is to use them in centrifuge modeling of soil-structure interaction (SSI) problems, where scales between 1:20 and 1:100 are typical. In such scales, the structure is only crudely modeled and typically it is attempted to preserve the similitude of stiffness using polymeric materials or aluminum alloys, while the strength is usually distorted, unless models with notches are used. Alternatively, small-scale reinforced concrete elements are constructed. To limit the concrete scale effects, used a model concrete comprising properly distributed sand as an aggregate and gypsum as a binding element. They manufactured the reinforcement by hand using steel wire. Manufacturing the reinforcement is the bottleneck both in terms of time to build the specimens and in terms of feasible scales. At a 1:40 scale, a 12-mm stirrup becomes 0.3 mm and this is clearly not manageable by hand.

The second reason is to use very small-scale physical models for the validation of the system-level (“global”) assumptions (e.g., global damping formulation, component interaction, boundary conditions) of the numerical models. These have been identified as a major source of error in numerical modeling in an opinion paper by Bradley. Indeed, blind prediction contests have shown that numerical models perform much better in predicting the component-level cyclic behavior of RC members than the system-level dynamic behavior of RC structures (Figure 1). This highlights the need for system-level shake table testing. In parallel, Refs. have shown that it is not always possible to validate numerical models deterministically (i.e., by trying to predict the response to individual ground motions). In fact, it is not even clear whether shake table tests of RC structures are repeatable. To this end, Refs. have proposed that models should be validated statistically under sets of ground motions, with a virgin specimen for each test. Clearly, this is not feasible in full scale, and the authors of this paper have suggested that model validation can be performed at a very small scale (say 1:40) and using a centrifuge.

It is acknowledged that a 1:40 model will be unavoidably distorted, as concrete properties scale with size, even when scaled aggregates are used. However, the purpose of this approach is not to create undistorted models but to statistically validate the global level assumptions (Figure 1) for given and experimentally obtained (at the model scale) component level behavior. Therefore, it needs to be clarified that the purpose of small-scale tests is not to determine the component level behavior of full-scale RC members. This can and should be determined by tests as close as possible to full scale. Moreover, the methodology discussed in this paper is easier to apply to modern structures that are designed to be ductile and fail by forming plastic hinges – a failure mechanism that is less influenced by scale phenomena. Physically modeling older structures that could fail in shear or because of the non-ductile design of joints is a larger step to take and is not the focus of this paper.

Notably, a 1:40 scale and the new ETH Zurich geotechnical centrifuge that has a platform of 1 × 1 m (extendable to 1 × 2 m) allows for testing of a typical five-story 15 × 15-m building or a whole overpass bridge together with the soil and the abutments.

Manufacturing of the 1:40 RC elements by hand presents three limitations: the construction time, the accuracy in positioning the reinforcement, and the size limitation of the steel rebars and stirrups. More specifically, manufacturing by hand the 1:50 bridge pier that was tested in Loli et al. takes 4 h of watchmaker’s precision work – and this is easier than manufacturing physical models of building columns that are even smaller. Clearly, it is not time-effective to build the reinforcement of a whole small-scale (e.g., 1:40) building by hand, while multiple specimens are impossible. Moreover, results from previous work show that hand accuracy might be good enough for SSI work, but it is not acceptable for studies focusing on the structure itself. Finally, it is not possible to manufacture the joints by hand, where column and beam rebars intersect with the stirrups. These limitations can be overcome by using a metal 3D printer to manufacture the reinforcement. The use of a metal 3D printer allows manufacturing of the entire reinforcement cage (longitudinal and horizontal reinforcement) with the desired layout.
This paper serves as a proof of concept and presents some first experimental results. It presents, for the first time, cyclic tests on 1:40 specimens with 3D printed reinforcement. It aims at characterizing the flexural behavior of 1:40 scale RC members by means of full-reversed quasi-static cyclic tests on cantilever specimens of different reinforcement ratios. A simple setup is used that only applies lateral load. The experimental results are compared to numerical simulations performed in OpenSees\textsuperscript{25} using modeling techniques commonly used with full-scale RC elements. The difficulties in 3D printing such small specimens and in performing cyclic tests at such scale are highlighted.

This work should not be confused with the attempts to use additive manufacturing technologies to construct full-scale buildings, either by 3D printing the concrete\textsuperscript{26–28} or the reinforcement.\textsuperscript{29,30} Both the means and the goals are completely different. The only similarity sources from the umbrella term “3D printing” that is used to describe very different technologies.

2 MATERIALS AND METHODS

2.1 Specimen description

The experimental program reported in the following sections consists of five cyclic cantilever tests of small-scale reinforced concrete members subjected to fully reversed cycles. The specimens are 55-mm height with a square cross section of $15 \times 15$ mm (Figure 2). Assuming a length scale of 1:40, in the prototype scale, the specimen has dimensions $600 \times 600 \times 2200$ mm. Assuming an inflection point in the mid-height of the member, the test is intended to study a $600 \times 600 \times 4400$-mm member. The base of the specimens consists of a square footing of 65 mm by 65 mm, 20-mm thick (Figure 2).

Two longitudinal reinforcement ratios are considered, that is, $\rho_l = 2.2\%$ and $\rho_l = 1.1\%$. The former will be referred to as high (H). The latter one will be referred to as low (L), not because the reinforcement is absolutely low, but because it is lower than the 2.2\% specimens. The H specimens have 16 bars of 0.6-mm diameter (24 mm in prototype scale) whereas the L specimens have 8 bars of 0.6-mm diameter. In both cases, the reinforcement is doubly symmetric with a concrete cover $c$ of 1.64 mm.
TABLE 1 Specimens list and details

| Label | Longitudinal reinforcement | Shear reinforcement |
|-------|----------------------------|--------------------|
| HH    | Heavier (H)                | High (H)           |
| HL    | Heavier (H)                | Low (L)            |
| LH    | Lighter (L)                | High (H)           |
| LL    | Lighter (L)                | Low (L)            |
| LN    | Lighter (L)                | No stirrups (N)    |

Regarding the transversal reinforcement, three layouts with different spacing were tested: 2.5 mm of spacing ($\rho_w = 0.76\%$, labeled H), 5 mm of spacing ($\rho_w = 0.38\%$, labeled L), and no transversal reinforcement (labeled with an N for no reinforcement). Shear ties are included in the specimens with transversal reinforcement. The diameter of the transversal reinforcement bars and the shear ties is 0.35 mm (14 mm in prototype scale).

Not all combinations of the above properties were tested. Table 1 summarizes the labeling and characteristics of each specimen.

### 2.2 Micro-concrete

Due to the small scale of the specimens, it is necessary to scale the inert component of the concrete so that the ratio inert size/rebars/cross section is about the same as in the prototype scale. Perth silica sand with a $d_{50} = 0.23$ mm was used. Its grain distribution compares well to the typical aggregate size when scaled 40 times (Figure 3).

The cement-based micro-concrete mixture consists of cement Normo52.5R, Perth sand, and water. The ratio binding component/sand/water was 1/1/0.5. The mix design was chosen to minimize the water percentage while preserving a good workability and flowability of mixture.

The mechanical properties of the micro-concrete were investigated through three uniaxial compression tests on cylindrical specimens of 20-mm diameter and 50-mm height, and three 4-point bending tests performed on prism specimens with cross section $15 \times 15$ mm and length of 80 mm. The size of the compression and 4-point bending test specimens was close to the size of the cantilever specimens, so that the values obtained from the material-level tests are directly applicable to modeling of the component-level tests. However, the specimens were larger than what would be dictated by a 1:40 scaling of the standardized specimens used for material testing of concrete (i.e., $300 \times 150$ mm cylinders and prisms of $40 \times 40 \times 160$ mm). The specimen tested in compression is roughly a 1:5 model of the standardized prototype cylinder and the one tested in 4-point bending is a 1:10 model of the standardized prototype prism – and this is one of the reasons for the high tensile to compressive strength obtained by the tests.

Figure 4 plots the stress–strain curves of the compression tests. The resulting average (coefficient of variation – CoV) compressive strength $f_c$ is equal to 34.9 MPa (3.4%), the strain at the maximum load $\varepsilon_1$ is $3.7\%$ (3.4%), and the modulus of rupture $f_{ct}$ is 4 MPa (5%). The Young’s modulus $E_c$ is calculated as the secant stiffness at 0.4 $f_c$ and it is equal to 13.4 GPa.
As only three tests were performed, the confidence interval on the CoV is relatively large and the values reported are only indicative.

### 2.3 3D printed reinforcement

The use of a 3D printer allows manufacturing of the entire reinforcement cage, namely the longitudinal and shear reinforcement, with the designed layout (Figure 5). Depending on the 3D printer, one can print rebars as small as 0.20 mm (200 μm), which in the prototype scale (1:40) represent Φ8 rebars. Even though significant advances were made in the recent years, additive manufacturing of metals is not a plug-and-play procedure yet and requires a careful preparation of the build job considering the geometry and size of the parts to achieve the best quality in a reasonable build time. In addition, overhanging features require support structures (Figure 5) to avoid part distortion and to prevent the part from local overheating. Support structures are printed in the same build job and need to be manually removed afterwards. The careful reader will observe that (a) the stirrups are completely closed (i.e., there is no 135° hook, as in the prototype elements), (b) the hooks of the shear ties are also closed, (c) there is a continuity between longitudinal reinforcement and stirrups, that is, they go through each other. This is a limitation sourcing from the capabilities of the metal 3D printer that was used, which, as all-metal 3D printers, cannot easily print overhanging features. The stirrups and shear ties being closed are not expected to distort the model, as modern buildings require hooks that are not supposed to open – and indeed they do not. The influence of the continuity between longitudinal and transverse reinforcement is quantifiable and can be taken into account when manufacturing physical models of specific target properties to be used in a centrifuge for system-level testing or for SSI problems, where the research question is not the behavior of the element, but of the whole system for a given component-level behavior.

This study used a Concept Laser M2 Laser Powder Bed Fusion (LPBF) printer that is able to manufacture various types of metal. A gas-atomized stainless steel 316L powder with a grain size 15–45 μm was used. All rebars were printed with ribs on the surface to increase bonding with concrete (Figure 6, left). The rib parameters for the 0.6-mm rebars (according
3D printed steel's stress–strain curves

![Figure 6](image)

The mechanical properties of the 3D printed steel were characterized with a series of uniaxial tension tests performed on 0.6-mm diameter bars identical to the ones used as reinforcement in the micro RC samples. Due to the small cross section, a sensor arm extensometer was used instead of strain gauges. The gauge length was 30 mm to measure the strains during the tests. The resulting average yield strength $f_y$ was 377 MPa (CoV = 13.2%). This value was calculated using the offset of 3‰ as suggested by ASTM E8/E8M-21. The average maximum strength $f_{y,max}$ was 417 MPa (12.1%) and the average Young’s modulus $E_s$ was equal to 176.7 GPa (1.6%). These values are in line with those reported by Casati et al. (2016) for 3D printed Stainless steel 316L. The experimental uniaxial behavior of the 3D printed steel is reported in Figure 6 (right): The stiffness of the printed material at such a small scale, that is, 0.6 mm of diameter, is consistent across multiple specimens. Nonetheless, the yield and ultimate stresses have a non-negligible variability. Given that larger 3D printed coupons that were tested showed less variability, the dispersion of Figure 6 (right) can be attributed to geometric imperfections at such a small scale. Therefore, in the future, either larger scales should be attempted, or finer steel powders (i.e., the raw material that the 3D printer uses) or more precise 3D printers should be used.

### 2.4 Experimental setup and instrumentation

The cyclic tests were performed in a universal testing machine (UTM), equipped with a support to attach the specimens and loading attachment. A fully reversed cyclic loading was applied at the top of the samples. The elements were placed horizontally and fixed on a steel support designed to fit in the UTM and to align the samples to the center of the machine (Figure 7). The base of the specimens was fixed to the lateral support with four M6 bolts. A $2 \times 65 \times 65$-mm steel plate was placed between the bolts and the face of the base to avoid local crushing of the concrete due to the compressive force applied by the screws.

The load is applied at the centerline of the element by two loading pins, which are connected to a double-hinged fork (Figure 7). The latter transfers the vertical load applied by the UTM. The measuring devices consist of two LVTD and a 3D-DIC system. The first LVDT measures the vertical displacement at the application point of the load, while the second LVDT measures the vertical displacement of the base of the element (to measure any possible sliding of the base). Digital image correlation (DIC) was used to measure the displacements and the strain field at the lateral surface of the RC member. The strain distribution was used to identify any cracks and micro-cracks formed during the tests and to locate the plastic hinges that formed at the base of the element.

### 2.5 Loading protocol

The load is applied by displacement control, applying a displacement $\Delta$ at the top of the elements. The kinematic parameter that is used to define the loading protocol is the drift ratio $\varphi$, which is defined as the ratio between the top displacement and the element length $l_e = 52.5$ mm (in this case it also represents the shear span).
The test program is defined based on FEMA-461,\textsuperscript{35} in which the amplitude increase is defined as $\varphi_{n+1} = 1.4 \times \varphi_n$. However, to capture the behavior of the element in the elastic range, the first amplitude was set equal to $0.5 \times \varphi_y$ and the second one was set to the yield drift, $\varphi_y$ (see Figure 8). The yield drift was calculated for each specimen using a fiber model in OpenSees\textsuperscript{25} and is defined as the drift that causes the first yield of reinforcement.

The velocity of application of the load was selected considering two principles. First, the displacement rate needs to be as low as possible to minimize any inertia and strain rate effects. Second, the loading rate needs to be reasonably fast to avoid creep and to make the test feasible time-wise. On these premises, the strain rate needed to be within a range of $\dot{\varepsilon} = 10^{-5} - 10^{-4} \text{ s}^{-1}$. This strain refers to the outermost fiber of the cross section of the RC member that lies closer to the foundation. The lower end of the strain-rate range corresponds to a displacement rate of the actuator equal to 0.075 mm/min, which was used for the first three amplitudes of the loading protocol. For the subsequent four amplitudes, the displacement rate was set to 0.413 mm/min, which corresponds to a strain rate of $5.6 \times 10^{-5} \text{ s}^{-1}$. According to Mander et al.,\textsuperscript{36} such a strain rate corresponds to an increase of concrete strength and stiffness of less than 3\% and 1.5\%, respectively, which is considered negligible. Finally, the last cycles were applied with a displacement rate of 0.75 mm/min, corresponding to the upper limit of the aforementioned strain rate range.

3 | RESULTS

3.1 | Load–deformation response: observations and discussion of the results

No shear failure occurred and all five specimens failed in bending. Concrete spalling (i.e., crushing of the unconfined concrete) occurred only in the HH and HL specimens. The spalling was not visible by naked eye and a magnifying lens
was needed. However, no concrete spalling occurred in specimens LH, LL, and LH. This is in contrast with the behavior of full-scale columns, but it can be explained by the lack of axial loading: the specimen loading conditions resemble more a beam than a column. In all cases, failure involved fracture of the longitudinal reinforcement. In all but the LL case, this occurred at the base cross section where a clearly visible crack was formed. In the LL case, failure was caused by a crack at a distance of 3.9 mm from the base. Micro-cracks were not visible by the eye because of the small scale of the specimens. However, DIC analysis (Figures 9 and 10, right: horizontal strain distribution at peak load, measured with 3D-DIC system) showed that micro-cracks along the length of the specimens did form, with the exception of the LL specimen. Based on the above, the behavior of the RC members is controlled by the behavior of the steel reinforcement.

Figures 9 and 10 (left) also offer the lateral force–displacement loops (P–D) for all tested specimens and the backbone curves for each specimen. The backbone is defined as the curve connecting the point of maximum displacement of the first cycle of each loading amplitude. The failure load \( F_{0.85} \) is conventionally defined as a strength degradation to 85% of the peak load \( F_{\text{max}} \) (i.e., of the strength). The results are summarized in Table 2. The relevant drift ratios are defined as \( \varphi_{F_{\text{max}}} \) and \( \varphi_{F_{0.85}} \).

Based on Figures 9 and 10, the following observations can be made:

(a) In all tests, there is a measured sliding on the order of 0.2 mm (=0.35%) in load reversal. This is a drawback of the custom-made clevis of the setup. In future tests, the displacement of the end cross section of the specimen itself should be directly measured. This setup drawback makes the relative error of the displacement at small drift ratios large. Therefore, this discussion will not focus on yield displacement but only on displacement and drift ratio at maximum load \( \varphi_{F_{\text{max}}} \) and ultimate displacement \( \varphi_{0.85} \).

(b) In the HH specimen, there is a clear offset of this sliding by roughly 30N indicating that for this test, there was a misalignment of the setup. This is reflected in not reaching maximum load towards negative displacements. So, the results for HH will not be furtherly discussed.
(c) A comparison of the HL specimen to the LH, LL, and LN specimens clearly shows that an increase in the longitudinal reinforcement causes an increase in strength. This behavior is expected and is compatible with the behavior of prototype RC members. A quantitative discussion on the issue is offered in the next section.

(d) A comparison of the HL and LL specimen reveals that an increase in longitudinal reinforcement causes an increase in both $\varphi_{\text{max}}$ and in $\varphi_{0.85}$. This is not compatible with the behavior in the prototype scale, as according to Panagiotakos and Fardis, the displacement at ultimate load $\varphi_{0.85}$ should not depend on the longitudinal reinforcement. However, given the variability in RC members response, especially when it comes to deformation, a comparison between only two specimens is not adequate for general conclusions and more specimens should be tested.

(e) A comparison of the LH, LL, and LN shows that the strength is not significantly influenced by the transverse reinforcement. This is compatible with the visual observation that these specimens failed because of the fracture
TABLE 2  Summary of the results from the cyclic tests

| Specimen | $F_{\text{max}}$ [N] | $\phi_{F_{\text{max}}}$ [%] | $F_{0.85}$ | $\phi_{0.85}$ [%] |
|----------|---------------------|-----------------------------|-----------|-------------------|
| HH       | +                   | 172.80                      | 1.474     | 146.88            | 3.002 |
|          | −                   | −                           | −         | −                 | −     |
| HL       | +                   | 169.60                      | 3.775     | 144.16            | 5.299 |
|          | −                   | 156.60                      | 2.660     | 132.60            | 3.263 |
| LH       | +                   | 83.7                        | 2.512     | 83.70             | 3.499 |
|          | −                   | 90.90                       | 2.286     | 77.26             | 4.291 |
| LL       | +                   | 93.66                       | 1.445     | 79.10             | 2.641 |
|          | −                   | 94.93                       | 1.415     | 80.69             | 3.408 |
| LN       | +                   | 86.70                       | 2.076     | 73.69             | 2.610 |
|          | −                   | 84.90                       | 0.830     | 72.16             | 3.258 |

of the longitudinal rebars and no concrete spalling was observed. Therefore, for the specimens tested, any possible increase of concrete strength because of confinement offered in LH and LL should not influence the strength of the RC member.

(f) From the above observations, it seems that specimens LH, LL, and LN should have behaved the same and the differences in their behavior can only be attributed to what can be called “natural variation,” that is variations caused by setup imperfections, or the geometric and mechanical properties of the materials. Biskinis, 38 based on 1844 tests published in literature, reports that such variations in prototype scale RC columns lie in between 0% and 38% for the yield moment and 0% and 59% for the yield drift. The CoV of $F_{\text{max}}$, $\phi_{F_{\text{max}}}$, and $\phi_{F_{0.85}}$ of the tests reported in this paper and assuming the positive and negative values as independent measurements are 5%, 37%, and 20%. Therefore, it can be concluded that for the small-scale specimens LH, LL, and LN, the “natural variation” observed is not something uncommon in full-scale tests too. This is in line with the observations of Knappett et al. 39

(g) Even though the specimens were symmetrically reinforced, their force–deformation loops are not perfectly symmetric. This is a behavior that is not incompatible with full-scale tests. For example, Saatcioglu and Ozcebe 40 report an asymmetric behavior for the specimen they tested under zero axial load. In the small-scale model, the asymmetric behavior can be explained by the variability of the steel properties, but also from the Bauschinger effect and the isotropic hardening of the steel under cyclic loading.

4 | NUMERICAL RESULTS

The purpose of this section is to show that numerical models developed for full-scale RC elements can describe the behavior of the tested small-scale models, if their parameters are calibrated against the small-scale material tests. Then according to the rationale developed in the introduction, these models could be used to validate the global level assumptions (Figure 1) against small-scale shake table tests that can be performed in a centrifuge.

To this end, numerical models of the cyclic tests were implemented in Opensees, and the results were compared against the experimental curves. The cantilever beam was modeled in 2D with three nonlinear forceBeamColumn elements and using a fiber model to characterize the hysteretic behavior of the rebars, unconfined concrete, and confined concrete (Figure 11). Each element included three integration sections.

The reinforcement was modeled using the Opensees Steel02 model (which is the Giuffré–Menegotto–Pinto model with isotropic hardening 41) enhanced with a MinMax model with strain at failure $\varepsilon_{\text{lim}}$. All but one of the parameters used were either calibrated on the uniaxial tension tests described in section 3.3 or the default Opensees values were chosen (Table 3). The sole parameter that was calibrated ad hoc on the cyclic tests of the RC member was $a_3$, which is a parameter that controls the isotropic hardening of steel.

The concrete was modeled using Concrete01 (which is the Kent–Scott–Park concrete model). 42 The parameters for the unconfined concrete (cover) were obtained from the material level tests on the small-scale specimens, while the $f_{\text{cc}}$, $\varepsilon_{\text{cc}}$, and $f_{\text{cucc}}$ of the confined concrete (core) were calculated using Mander’s model that provides the confined concrete stress–strain curve for given unconfined concrete properties and transverse reinforcement. 36 As Mander’s model does not
FIGURE 11  Schematic of the OpenSees model using fiber model and nonlinear forceBeamColumn elements

| Parameter | Value | Parameter | Value |
|-----------|-------|-----------|-------|
| $f_y$     | 377.8 [MPa] (tests) | a1 | 0 (default) |
| $E_s$     | 177 [GPa] (tests) | a2 | 1 (default) |
| $b_s$     | 0.003 (tests) | a3 | 0.02 |
| R0        | 15 (tests) | a4 | 1 (default) |
| cR1       | 0.925 (default) | $\varepsilon_{lim}$ | 0.135 (tests) |
| cR2       | 0.15 (default) | |

TABLE 3  Steel02 parameters

| Specimen | Unconfined $f'_c$ [MPa] | $\varepsilon_c$ | Confined $f_{cc}$ [MPa] | $\varepsilon_{cc}$ | $f_{ccu}$ [MPa] | $\varepsilon_{ccu}$ |
|----------|--------------------------|-----------------|------------------------|-------------------|----------------|-------------------|
| HH       | 34.5                     | 0.0038          | 43.11                  | 0.008             | 38.40          | 0.059             |
| HL       | 34.5                     | 0.0038          | 38.26                  | 0.005             | 23.64          | 0.048             |
| LH       | 34.5                     | 0.0038          | 43.11                  | 0.008             | 38.40          | 0.059             |
| LL       | 34.5                     | 0.0038          | 38.26                  | 0.005             | 23.64          | 0.048             |
| LN       | 34.5                     | 0.0038          |                       |                   |                |                   |

TABLE 4  Concrete01 parameters of cover and core concrete

provides the ultimate stress $\varepsilon_{ccu}$, this was computed using the formula suggested by CEN-fib Model Code. All parameters are summarized in Table 4 and defined in Figure 12.

An additional zero-length section element was used at the fixed end of the cantilever beam in order to model strain penetration. The strain penetration causes slippage of the anchored bars, which leads to a fix-end rotation of the beam-column element. The fix-end rotation can be captured modeling the steel fiber in the zero-length section with the Bond_SP01 model developed by Zhao and Sritharan, which accounts for the bar slippage. The model used by the Bond_SP01 element is defined by six parameters; yield stress of the rebars $F_y$, ultimate stress of the rebars $F_u$, a parameter $R$ that governs the pinching of the cyclic force-slip loops, the hardening ratio $b$ of the force-slip loops, and $s_y$ and $s_u$ which represent the rebar slip at yielding and the rebar slip at failure, respectively. Zhao and Sritharan defined $s_y$ as

$$s_y = 2.54 \left( \frac{d_b \cdot (mm)}{8437} \cdot \frac{f_y (MPa)}{\sqrt{f'_c}} \cdot (2\alpha + 1) \right)^{1/\alpha} + 0.34$$

which was derived from a set of pullout tests at full-scale – consequently it is not suitable for small-scale models. Preforming small-scale pullout tests to determine $s_y$ lies beyond the scope of this paper. Therefore, $s_y$ was calibrated so that the experimental curves of the cyclic tests discussed in the previous section match the OpenSees numerical results. The value for $s_y$ is conventionally defined as $40s_y$. The Bond_SP01 model parameters are reported in Table 5.

Figure 13 compares the experimental and numerical results of all the samples. With the exception of the last cycles, the numerical model is able to capture the cyclic loops with a reasonable accuracy. Notably, as these are cyclic tests, the displacement protocol is used as an input to the numerical model that essentially predicts the corresponding forces. A drawback of the model is that the longitudinal reinforcement in the numerical model did not fracture and this explains the
TABLE 5  Bond_SP01 parameters

| Parameter | Value  |
|-----------|--------|
| $f_y$     | 377.8  |
| $B$       | 0.5    |
| $R$       | 1      |
| $s_y$     | 0.03   |
| $s_u$     | 1.2    |

The good match between the experimental results of cement-based micro RC models and the Opensees models, calibrated with standard material models, which are generally used in full-scale applications, suggest that small-scale physical models manufactured with 3D printed reinforcement can be used to perform system-level testing of whole structures with the purpose of obtaining datasets that can be used for the statistical validation of system-level assumptions. However, more research is needed in order to accurately physically model strain penetration.

5  CONCLUSIONS

A large part of uncertainty and error in numerical modeling lies in the global level assumptions of numerical models. This has not attracted the attention it deserves, because it is not easy to produce much system-level (i.e., structure-level) data.

This paper proposes testing small-scale structures (e.g., whole buildings) on a shake table placed in a centrifuge. Sand can be used as an aggregate for small-scale concrete and submillimeter diameter reinforcing steel can be printed and placed by a metal 3D printer. Model (“small scale”) concrete showed similar compressive and flexural strength with prototype concrete. The 0.6-mm diameter 3D printed rebars had a Young Modulus of 177 GPa and a yield stress on the order of 380 MPa, making it similar (albeit slightly weaker) to steel used for reinforcement. However, the dispersion of its strength is larger at such small diameters – something that is not observed in larger coupons.

RC members having cross section as small as $15 \times 15$ mm and reinforcement of 0.6 mm (longitudinal) and 0.35 mm (transverse) were manufactured and tested cyclically. They were designed to fail in bending by failure of their reinforcement. The RC members were tested cyclically and their behavior was similar to the behavior of full-scale specimens, in terms of their hysteresis loops. For these specific cases, their variability was similar to the variability observed in full-scale tests.
An OpenSees model, using elements and materials developed for full-scale structures, was built. The model used fiber elements and took account of strain penetration. It was able to capture the experimental behavior with a reasonable accuracy.

In the future, more tests are needed. At the material level, the steel behavior needs to be better quantified via cyclic tests. The steel–concrete bond behavior needs to be tested and made similar to the prototype behavior by fine tuning the roughness and ribs of the printed rebars. At a component level, tests with more longitudinal reinforcement and denser stirrups should be tested to study the behavior of confined cross sections where the concrete properties govern their flexural response.
It seems feasible to produce small-scale models of a full structure to perform dynamic tests in a geotechnical centrifuge. The shake table tests could provide datasets to statistically validate the global level assumptions that are usually made to scale up from component- to system-level behavior. Moreover, physical modeling or RC at such small scale could provide experimental data for problems on which few physical tests have been performed, like pounding of buildings or bridge–abutment interaction.

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REFERENCES
1. Harris HG, Sabnis G. Structural Modeling and Experimental Techniques. Boca Raton, FL: CRC Press; 1999.
2. Little WA, Paparoni M. Size effect in small-scale models of reinforced concrete beams. J Am Concr Inst. 1966;63(11):191–1204.
3. Bažant ZP, Kazemi MT. Size effect on diagonal failure of beams without stirrups. ACI Struct J. 1991;88(3):268–276.
4. Bažant ZP, Li Z. Modulus of rupture: size effect due to fracture initiation in boundary layer. J Struct Eng. 1995;121(4):739–746.
5. Belgin CM, Sener S. Size effect on failure of overreinforced concrete beams. Eng Fract Mech. 2008;75:2308–2319.
6. Knappett JA, Reid C, Kinmond S, O’Reilly K. Small-scale modeling of reinforced concrete structural elements for use in a geotechnical centrifuge. J Struct Eng. 2011;137(11):1263–1271.
7. Abdoun T, Doby R, O’Rourke TD, Goh SH. Pile response to lateral spreads: centrifuge modeling. J Geotech Geoenvironmental Eng. 2003;129(10):869–878.
8. Hayward T, Lees AS, Powrie W, Richards DJ, Smethurst J. Centrifuge Modelling of a Cutting Slope Stabilised by Discrete Piles. 2000.
9. Knappett JA, Madabhushi SPG. Influence of axial load on lateral pile response in liquefiable soils. Part I: physical modelling. Geotechnique. 2009;59(7):571–581.
10. Deng L, Kutter BL, Kunnath SK. Centrifuge modeling of bridge systems designed for rocking foundations. J Geotech Geoenvironmental Eng. 2012;138(3):335–344.
11. Loli M, Knappett JA, Brown MJ, Anastasopoulos I, Gazetas G. Centrifuge modeling of rocking-isolated inelastic RC bridge piers. Earthq Eng Struct Dyn. 2014;43(15):2341–2359.
12. Al-Defae AH, Caucis K, Knappett JA. Aftershocks and the whole-life seismic performance of granular slopes. Géotechnique. 2013;63(14):1230–1244.
13. Al-Defae AH, Knappett JA, (2014). Stiffness matching of model reinforced concrete for centrifuge modelling of soil-structure interaction. In Proceedings of the 8th International Conference on Physical Modelling in Geotechnics, ICPMG 2014. CRC Press; 2014:1067–1072.
14. Al-Defae AH, Knappett JA. Centrifuge modeling of the seismic performance of pile-reinforced slopes. J Geotech Geoenvironmental Eng. 2014;140(6):04014014.
15. Bradley BA. A critical examination of seismic response uncertainty analysis in earthquake engineering. Earthq Eng Struct Dyn. 2013;42(11):1717–1729.
16. Trüb M. Numerical modeling of high performance fiber reinforced cementitious composites. IBK Bericht. 2011;333.
17. Collapse Prevention Center. 2011. http://www.collapse-prevention.net/download/Competition/RC_Frame/RC_Frame_Competition.htm. Accessed September 29, 2019.
18. Lin X, Lu X. Numerical models to predict the collapse behavior of RC columns and frames. Open Civ Eng J. 2017;11(1):854–860.
19. Schoettler MJ, Restrepo JI, Guerrini G, Duck DE, Carrea F. A Full-Scale, Single-Column Bridge Bent Tested by Shake-Table Excitation [PEER Report 2015/2]. 2015:1–122.
20. Terzic V, Schoettler MJ, Restrepo JI, Mahin SA. Concrete Column Blind Prediction Contest 2010: Outcomes and Observations [PEER Report 2015/1]. 2015:1–145.
21. Bachmann JA, Strand M, Vassiliou MF, Broccardo M, Stojadinovic B. Is rocking motion predictable. Earthq Eng Struct Dyn. 2018;47(2):535–552.
22. Bachmann J, Strand M, Vassiliou MF, Broccardo M, Stojadinovic B. Modelling of rocking structures: are our models good enough?. In Proceedings of the 2nd International Conference on Natural Hazards & Infrastructure (ICONHIC 2019). 2019.
23. Del Giudice L, Vassiliou MF. Mechanical properties of 3D printed material with binder jet technology and potential applications of additive manufacturing in seismic testing of structures. Additive Manuf. 2020;36:101714.
24. Del Giudice L, Wrobel R, Leinenbach C, Vassiliou MF, Static testing of additively manufactured microreinforced concrete specimens for statistical structural model validation at a small scale. In Proceedings of the 8th International Conference on Advances in Experimental Structural Engineering (SAESE), Christchurch, New Zealand, 3–5 February. 2020.
25. Mazzoni S, McKenna F, Scott MH, Fenves GL. OpenSees Command Language Manual. University of California, Berkeley: Pacific Earthquake Engineering Research (PEER) Center; 2006.
26. Khoshnevis B. Automated construction by contour crafting—related robotics and information technologies. *Autom Constr.* 2004;13(1):5–19.
27. Asprone D, Auricchio F, Menna C, Mercuri V. 3D printing of reinforced concrete elements: technology and design approach. *Constr Mater*. 2018;165:218–231.
28. Buswell RA, De Silva WL, Jones SZ, Dirrenberger J. 3D printing using concrete extrusion: a roadmap for research. *Cement Concrete Res.* 2018;112:37–49.
29. Hack N, Wangler T, Mata-Falcón J, et al. Mesh mould: an on site, robotically fabricated, functional formwork. In *Proceedings of the Second Concrete Innovation Conference (2nd CIC)*. 2017;19:1–10.
30. Mechtcherine V, Grafe J, Nerella VN, Spaniol E, Hertel M, Füssel U. 3D-printed steel reinforcement for digital concrete construction – manufacture, mechanical properties and bond behaviour. *Constr Building Mater.* 2018;179:125–137.
31. Tanaka H, Park R. *Effect of Lateral Confining Reinforcement on the Ductile Behavior of Reinforced Concrete Columns* [Report 90-2]. Department of Civil Engineering, University of Canterbury; 1990.
32. EN:10080, Steel for reinforcement of concrete – weldable reinforcing steel – general. 2005.
33. ASTM E8/E8M-21. *Standard Test Methods for Tension Testing of Metallic Materials*. West Conshohocken, PA: ASTM International; 2021.
34. Casati R, Lemke J, Vedani M. Microstructure and fracture behavior of 316L austenitic stainless steel produced by selective laser melting. *J Mater Sci Technol*. 2016;52(8):738–744.
35. FEMA A, 461. *Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Nonstructural Components*. Applied Technology Council, Redwood City, CA; 2007;113
36. Mander JB, Priestley MJ, Park R. Theoretical stress-strain model for confined concrete. *J Struct Eng*. 1988;114(8):1804–1826.
37. Panagiotakos TB, Fardis MN. Deformations of reinforced concrete members at yielding and ultimate. *Struct J*. 2001;98(2):135–148.
38. Biskinis DE. *Resistance and Deformation Capacity of Concrete Members with or Without Retrofitting* [Doctoral thesis]. Civil Engineering Department, University of Patras, Patras; 2007.
39. Knappett JA, Brown MJ, Shields L, Al-Defae AH, Loli M, (2018). Variability of small scale model reinforced concrete and implications for geotechnical centrifuge testing. In *Proceedings of the 9th International Conference on Physical Modelling in Geotechnics*. 241–246.
40. Saatcioglu M, Ozcebe G. Response of reinforced concrete columns to simulated seismic loading. *Struct J*. 1989;86(1):3–12.
41. Filippou FC, Popov EP, Bertero VV. Report EERC 83-19. Berkeley, CA: Earthquake Engineering Research Center, University of California, Berkeley; 1983. *Effects of Bond Deterioration on Hysteretic Behavior of Reinforced Concrete Joints*. 1983.
42. Scott BD, Park R, Priestley MJN. Stress–strain behavior of concrete confined by overlapping hoops at low and high strain rates. *Am Concr Inst J*. 1982;79(1):13–27.
43. *Fib*. (2012). *Model Code 2010 – Final Draft*. Vol. 2. Berlin: Ernst & Sohn: The International Federation for Structural Concrete, Bulletin 66
44. Zhao J, Sridharan S. Modeling of strain penetration effects in fiber-based analysis of reinforced concrete structures. *ACI Mater J*. 2007;104(2):133–141.

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