A Low-Complexity Candidate for Benchmarking Collapse-Prediction of Steel Braced Structures

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ABSTRACT

To aid in the evaluation of the collapse-prediction capability of competing methodologies, a case study of a water-tank subjected to the Takatori near-source record from the 1995 Kobe earthquake, scaled down by a factor of 0.32, is presented. The water-tank, supported by a 5-segment steel lattice tower, is so configured as to have a characteristic collapse mechanism that is triggered due to catastrophic column and brace buckling at the bottom-most segment of the lattice under all forms of ground motion. A FRAME3D model of the tank reveals severe buckling in the bottom mega-columns and one of the two braces on the west face of the tower when the structure is impacted by the Takatori near-source pulse, resulting a tilt in the structure. This is followed by sequential compression buckling of braces on the south and north faces leading to $P - \Delta$ instability and complete collapse of the tank. In order to verify the predictions of the FRAME3D model, a comparable PERFORM-3D model of the tank, using fiber elements and constitutive material models that are suitably calibrated against experimental data, is developed. The response of this model to the scaled Takatori ground motion compares very well against that of the FRAME3D model; the smallest scaling factor needed to collapse the PERFORM-3D model is 0.323, whereas the corresponding factor needed to collapse the FRAME3D model is 0.315. The sequence of column- and brace-buckling failures and the collapse mechanisms are quite similar in the two models.

Introduction

Numerical modeling of collapse is one of the grand challenges in the structural and earthquake engineering fields. A remarkable number of techniques have evolved in this area in the last decade or so and the algorithms capable of predicting collapse have gotten quite sophisticated. Unfortunately, verification and validation aspects have not received as much attention, possibly because of the lack of problems that are suitable for testing. The problems for which analytical solutions can be written are too simple and do not quite test all features of the algorithms. Full-scale experiments on assembled structures are starting to become more common and will likely be the future test-beds for the algorithms. In the mean time, there is a need for some standard benchmark problems that can be used to test and compare the algorithms.

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Existing benchmark problems in structural engineering have been developed by the structural health monitoring and control community for convenient evaluation of control algorithms and strategies. Three generations of steel benchmark buildings subjected to wind and seismic excitation [21, 20], one benchmark problem for base-isolated structures [18, 17, 19], one benchmark problem for highway bridges [1], and one for cable-stayed bridges [5], have been developed. The current generation of benchmark buildings consist of three typical steel structures (3-, 9-, and 20-story buildings) modeled in two dimensions using bilinear moment-curvature relations to describe beam- and column-end behavior. Geometric nonlinearity (second-order $P - \Delta$) effects are not included in these models at the element level (governing member buckling) or the assembled structure level (governing global stability). As a result, these models are not designed to capture collapse and cannot be used for collapse benchmarking. Moreover, three-dimensional analysis is generally needed to accurately predict collapse. Collapse prediction under strong earthquake ground motion is a fast-evolving field, with many different methodologies being developed currently, and there is an urgent need for benchmark problems that can be used for the evaluation of these methods. In the aftermath of the Northridge earthquake, SAC, a consortium of the Structural Engineers Association of California, the Applied Technology Council, and the Consortium of Universities for Research in Earthquake Engineering, commissioned three consulting firms to design three other 3-, 9-, and 20-story index buildings, conforming to local code requirements for Los Angeles, Seattle, and Boston. Many researchers have analyzed these buildings under earthquakes to characterize response under various hazard levels (e.g., [6, 7]). In earthquake engineering, these structures have become de facto benchmark structures in as far as moment frame buildings are concerned. To the best of the authors’ knowledge, no such models exist for braced structures.

Benchmark problems for collapse of braced structures must be formulated at multiple levels of complexity in order to systematically evaluate the capabilities of collapse prediction methodologies. In the authors’ experience, subtle changes in material models, including hysteretic rules, can lead to substantially different member failure sequences and structure collapse mechanisms. It would help if the first generation benchmark problem is of low complexity where the collapse mechanism is not overly sensitive to these subtle changes in the material model; perhaps the collapse mechanism is known or can be inferred from basic structural mechanics principles, yet the actual phenomenon of collapse is quite involved. The water-tank presented here (Figure 1) is a fictional structure that is configured such that if collapse occurs it is always triggered by element buckling in the same region of the supporting lattice when excited by any ground motion. It has not been designed according to any code, however members were proportioned to be able to carry the self-weight of the tank and the water without yielding. Given the characteristic nature of its collapse mechanism, this structure could serve as a good candidate for low-complexity benchmarking. The benchmarks for the verification of competing collapse-prediction
methodologies could include the collapse mechanism itself, the onset of buckling in the various members leading to global collapse, the post-buckling behavior of columns and braces (hysteretic loops), period-lengthening observed in horizontal force resultants, the tank roof displacement and trajectory as it collapses, time beyond which the algorithm is unable to track the progression of collapse, and execution time on a personal computer. Future generations of benchmarking may involve higher levels of complexity and could test progressively harder
collapse modeling features such as partial fixity of connections, foundation compliance, soil-structure interaction, modeling uncertainty, construction quality uncertainty, out-of-plane buckling of the diagonal X bracing as a result of being connected at the brace intersection point, low-cycle fatigue and fracture, etc., as well as structures with multiple global collapse modes.

**Modeling of the Water Tank**

The tank is 160’ (48.76 m) high and has a capacity of 66390 cu.ft. (1880 cu.m.). The total weight of the structure is 6248 Kips (27792 kN) which includes a full tank of water, a 16” (406 mm) thick concrete floor slab, a 12” (305 mm) thick concrete roof slab, a 12” (305 mm) thick concrete tank wall, and the weight of the steel frame. The supporting lattice consists of five 320” (8128 mm) segments (labeled 1–5 in Figure 1) over its height, with four sloping mega-columns interconnected by beams and X-braces forming a rigid spine. In addition to these elements in near-vertical planes, horizontal diagonal elements (X-braces) are provided at 320” (8128 mm) height intervals. They serve to provide lateral stiffness and strength for wind and earthquake resistance, as well as lateral support to the four columns. The four mega-columns are made of B30x30x0.625 tube sections (the non-standard symbol “B” is used to indicate that nominal dimensions rather than actual “TS” dimensions listed in the AISC manual of steel construction are used in the model) with depth and width of 30” (762 mm), and wall thickness of 0.625” (15.875 mm); the beams and braces are made of B14x14x0.625 tube sections; and the horizontal diagonals are made of B12x12x0.5 tube sections. The horizontal and vertical X-braces are assumed to be disconnected at the intersection points (and hence there is no restraint to in-plane or out-of-plane buckling at these points). Providing a connection at the mid-points would result in increased complexity in buckling behavior and make response prediction much more challenging. The water-tank floor and roof are two-way slabs that are supported by two trusses in either direction. The elements of the truss (floor and roof beams that form the top and bottom chords of the trusses, perimeter stub columns, and vertical braces) are made of B20x20x0.625 tube sections. All sections are assumed to be of ASTM-A501 steel with a yield stress of 46 ksi (317.16 MPa) and an ultimate stress of 58 ksi (399.90 MPa). Full continuity is assumed at all the connections and a fixed boundary condition is assumed for the base of the water-tank. Due to structural symmetry, the periods corresponding to the two orthogonal fundamental translational modes is 1.31 s, whereas the period of the first torsional mode is 0.72 s. If the mass of the members is neglected and all the mass is assumed to be concentrated in the tank portion of the structure, then the structure can be idealized as a 3 degree-of-freedom system (mass translation in X, Y and Z directions). The motion of the mass would lead to overturning moments that are greatest at the base of the tank. So the mega-columns at the bottom are the most stressed. Even though the outward flare in the columns increases the lever arm to resist the overturning moment, it is not sufficiently...
large to make up for the increase in the overturning moment over the height of the tower. At the same time, the bottom columns have slightly longer clear spans when compared to the upper columns. This is because the upper columns have finite-sized joints at both their ends, whereas the bottom columns have finite-sized joints only at the top, with the bottom end being connected to a base plate at zero elevation (see Figure 1). The sloping columns also result in longer spans and shallow angles for the bottom braces. Maintaining the size of all columns and braces constant would then ensure that the collapse of the tank always occurs due to column and brace buckling in the bottom segment (Segment 1) and subsequent overturning due to $P-\Delta$ effects.

This inference of a characteristic collapse mechanism, based on structural mechanics principles, can be verified using a numerical model of the tank and studying its response under several collapse-causing ground motion records. Toward this end, a FRAME3D [10] model of the tank using modified elastofiber (MEF) elements [11, 13] for all the steel members $[\sigma_y = 46$ ksi (317.16 MPa), $\sigma_u = 58$ ksi (399.90 MPa), $E_{sh} = 580$ ksi (3998.96 MPa), $\epsilon_{sh} = 0.012, \epsilon_u = 0.16$, cross-sectional minor and major direction eccentricity of $5 \times 10^{-6}$ L] is developed.

MEF beam elements are subdivided into 5 segments, two nonlinear segments at the ends and one nonlinear segment in the middle, with elastic segments sandwiched between the nonlinear segments [Figure 2(a)]. Each nonlinear segment is further discretized into 20 fibers that run the full length of the segment. Associated with each fiber is a nonlinear hysteretic stress-strain law [8] for axial stress, $\sigma_n$, and axial strain, $\epsilon_n$, where $n$ denotes the $n^{th}$ fiber [Figure 2(b)]. The backbone curve consists of a linear portion, a yield plateau, a strain-hardening region which is described by a cubic ellipse, and a strain-softening region described by a continuation of the same cubic ellipse culminating in fiber rupture. The backbone curve is characterized by seven parameters: yield stress, $\sigma_y$, ultimate stress, $\sigma_u$, Young’s modulus, $E$, strain at initiation of strain hardening, $\epsilon_{sh}$, strain at ultimate stress, $\epsilon_u$, rupture strain, $\epsilon_r$, and the tangent modulus at initiation of strain hardening, $E_{sh}$. Hysteresis loops consist of linear segments and cubic ellipses, and the hysteretic rules to define the cyclic response of each fiber are given by [3]. While low-cycle fatigue is not explicitly included, FRAME3D does allow for a user-specified probabilistic description of the fracture strain of fibers in MEF elements, and fibers can fracture at randomly selected strain levels. However, because it is important for benchmarking results to be unique, deterministic, and reproducible, this feature is not used in the water-tank model. Element local failure can still occur in the form of fiber rupture, culminating in member severing if all fibers in a segment rupture. Coordinates of model nodes, including the interior nodes of the MEF elements, are updated during each iteration of a single analysis time-step, and dynamic equilibrium is satisfied in the updated configuration. This automatically accounts for local $P-\Delta$ effects such as member buckling as well as global $P-\Delta$ effects, and allows the analysis to follow a structure’s response well into collapse. Local buckling is not included in the FRAME3D formulation, however,
Figure 2: (a) Layout of the five-segment modified elastofiber element (fiber arrangement is shown for an I-section and a box section). (b) Axial stress-strain hysteresis model for each fiber.

and is omitted in this case study as well.

Joints are modeled using specially designed cruciform elements in FRAME3D [15]. The cruciform section is formed by two planar orthogonal panels. Vertical edges of these panels contain attachment points where beams attach, and horizontal edges on the top and bottom, where columns attach (Figure 3). Braces attach to the corner points of panels. This accounts for the finite size of joints. Either panel may yield and strain harden in
shear. However, panels in this model are assumed rigid and panel zone deformation is neglected, limiting model complexity. Rayleigh damping coefficients are computed with a desired amount of 2% damping at periods of 5.0 s and 0.5 s. Although the fundamental period of the tank is 1.31 s, the period may lengthen significantly as the supporting structure gets damaged, hence the choice of 5 s and 0.5 s to constrain the Rayleigh coefficients. FRAME3D uses the initial stiffness matrix in determining the damping matrix. A time-step size of 0.005 s is used for the ground motion analyses. The Constant Average Acceleration method (Newmark’s method with $\gamma = 0.50$ and $\beta = 0.25$) is used for time integration. The concrete roof and floor slab of the tank are modeled using plane-stress elements [Young’s modulus, $E = 3605$ ksi ($24855.60 \text{ MPa}$), and Poisson’s ratio, $\nu = 0.20$]. The model consists of 44 node points, 144 MEF elements, and 18 plane-stress (diaphragm) elements. Mass is lumped at the node points (see sections on Figure 1).

To verify the characteristic collapse mechanism of the water tank, its FRAME3D model is subjected to 13 three-component records from the 21 September 1999, magnitude 7.7 Chi-Chi earthquake in Taiwan, and the 25 September 2003, magnitude 8.3 Tokachi-Oki earthquake in Japan, scaled by a factor of 3. Response spectra of the north and east components of the unscaled version of these records are given in [16]. The model
collapses in all but three cases (scaled records at stations HKD100, HKD112, and HKD113, from the Tokachi-Oki earthquake). To isolate the region with the greatest damage, the relative vertical (Z) displacement of the top and bottom of each segment at the north-west corner normalized by the segment height is shown plotted in Figure 4. It is common to track lateral (X and Y) drifts to identify sway collapse mechanisms. However, collapse mechanisms resulting from asymmetric column buckling will lead to rigid-body tilting of the structure above causing large (X and Y) sway drifts in the upper structure even though collapse is being triggered within a lower segment. Tracking X or Y sway drift to identify the collapse mechanism could thus be misleading. Vertical (Z) drift, on the other hand, will be large for the segment with column buckling and small for the upper segments that undergo rigid-body tilting resulting from settlement in the lower segment due to asymmetric column buckling, correctly identifying the collapse mechanism. Z drifts using displacements at each of the four mega-column locations [12] indicate that most of the relative vertical deformation in the water-tank is concentrated in the bottom segment (Segment 1). In all instances of collapse, the upper structure simply undergoes rigid-body tilting and free-fall caused by asymmetric column and brace buckling in Segment 1.

**Incremental Dynamic Analysis Using the Takatori Record**

The Takatori station near-source record from the January 17, 1995, $M_w = 6.9$ Kobe earthquake (Figure 5), scaled suitably to trigger collapse of the water-tank, is selected for benchmarking purposes. The strong com-
Figure 5: Takatori station near-source record (unscaled) from the January 17, 1995, $M_w = 6.9$ Kobe earthquake (acceleration, velocity, and displacement) and 5% damped pseudo-acceleration response spectra with the translational and torsional periods of the water-tank indicated for reference.

The component of this record has a spectral peak at a period very close to the fundamental period of vibration of the water-tank (1.31 s). Incremental dynamic analyses are performed on the water-tank model using this record to establish the minimum scaling factor needed to cause collapse. The three-component record is scaled down by a factor ranging between 0.04 and 1.00, at 0.04 increments, and used to excite the FRAME3D model of the tank. Shown in Figure 6 are the global X, Y, and Z direction peak absolute resultant forces in each of the five segments of the lattice supporting the tank as a function of the scaling factor. There is a monotonic increase in the peak lateral (X and Y) forces carried by all the segments up to a scale factor of 0.32. Beyond this, the stiffness forces saturate, suggesting that this is the collapse threshold for the tank. A more precise threshold for collapse of the FRAME3D model is a scaling factor of 0.315. However, a scaling factor of 0.32 is used for collapse benchmarking here.
Figure 6: Peak absolute resultant forces in each of the five segments of the water-tank under incremental dynamic analysis using scaled Takatori records: (a) Global X direction. (b) Global Y direction. (c) Global Z direction (vertical).

Peak drifts at the north-west corner in the X, Y, and Z directions as a function of the ground motion scaling factor are shown plotted in Figure 7. Beyond the scaling factor of 0.32, the Z drift in Segment 1 is distinctly different from that in the upper segments. The relative Z deformation (drift) between the top and bottom of segments 2-5 is minor when compared to that of Segment 1 [Figure 7(c)], demonstrating that most of the downward displacement as the tank collapses is the result of damage in the bottom-most segment. Column buckling in the SW and NW columns of Segment 1, along with brace buckling on the northern, southern, and western faces of the segment, causes differential “settlement” within Segment 1 [Figures 8(a) and 8(b)]. This results in a tilt of the upper structure inducing a large $P - \Delta$ moment. The X and Y lateral drifts [Figures 7(a) and 7(b)] are caused not just by lateral sway, but also by this tilting of the tank and the supporting tower as a rigid body about the point of damage concentration. Thus, above this “hinge zone”, the relative deformations between the top and bottom of each of the segments in either direction due to rigid body rotation are nearly identical, confirming that these segments are essentially undamaged. Based on these observations, it can be concluded that all the damage leading to collapse of the tank is concentrated in the bottom-most segment. The structure collapses in a mixed mode [Figure 8(c)], combining the vertical, but asymmetric, free-fall of the upper structure.
through the bottom segment, with the overturning of the upper structure. Drifts at the south-west, south-east, and north-east corners can be found in [12].

**Benchmarking Using the Takatori Record Scaled by a Factor of 0.32**

For benchmarking purposes, detailed results from the analysis of the FRAME3D model of the tank subjected to the Takatori record scaled down by a factor of 0.32 are presented in [12]. At about 5.6 s into the record (Figure 5), the forward phase of the largest near-source pulse initiates buckling, though just barely, in the bottom mega-column at the north-east corner [Figure 9(b)]. Before buckling can progress, ground velocity changes direction and the structure is stabilized. However, the reverse phase of the same ground motion pulse induces severe buckling on both bottom mega-columns on the west face of the tower [Figure 9(a)], followed almost instantaneously by buckling on one of the braces on the west face of the bottom segment [Figure 10(b)]. Multiple fibers rupture in the two buckled mega-columns starting at 6.34 s; fibers in the middle segment rupture first,
Figure 8: Snapshots of the collapsing steel braced water-tank FRAME3D model subjected to the 1995 Kobe earthquake Takatori record scaled down by a factor of 0.32. Deformations are exaggerated by a factor of 5.

followed by fibers in the end segments. The loss in compression load-carrying capacity of the two columns results in their load being transferred to the braces of Segment 1, and they do not experience tension for the remaining duration of shaking [Figure 10(c)]. As shaking progresses the braces on the south and the north faces of the tower buckle sequentially at 9.6 s, 12.5 s, and 14.35 s [Figure 10(b)]. The concurrent loss of vertical load- and lateral force-resisting capacity due to buckling of the braces results causes the water-tank to lean to one side. $P - \Delta$ effects trigger collapse of the tank [Figure 8(c)]. The program is able to track the collapse of the structure until time 16 s.

It is hard to deduce that all the damage is restricted to Segment 1 from the global X and Y direction drift histories [12]. Since the structure collapses under significant overturning, the lateral drifts in the upper segments are not insignificant. This is not the case with the “vertical (Z) drift”. The Z direction drifts for Segment 1 far exceed that of the upper segments (Figure 11), indicating that significant buckling is occurring in this segment, causing the upper structure to undergo free-fall while remaining virtually intact. Unlike the drift response histories where there are no zero crossings as the structure collapses, period lengthening can be observed in the global X and Y force resultant histories of all the segments [Figure 12(a)]. The EW period lengthens from 1.31 s to about 2 s, whereas the NS period lengthens from 1.31 s to about 1.7 s. The segment global Z force histories shown in Figure 12(b) are also instructive. When the water-tank is stable, the only differences in vertical forces carried in all the five segments of the supporting structure arise from the self-weight of the frame. This holds true until about 15.5 s into the record, at which point there is sudden and swift degradation of the vertical load carrying capacity of Segment 1, and it is not able to transfer the load imposed from above to the ground, resulting
Figure 9: Time histories of (a) minor and major direction lateral deformation and (b) axial forces in the base (Segment 1) mega-columns at the four corners (south-west: sw, south-east: se, north-west: nw, and north-east: ne) of the water-tank under the Takatori ground excitation scaled down by a factor of 0.32. (c) Column axial forces plotted as a function of column lateral deformation.

in eventual free-fall of the upper structure.

The displacement response histories of the SW and NE corners of the roof are given in Figure 13. The peak displacements of the SW corner at the time beyond which FRAME3D is unable to track the collapse of the tank are -66.7” (-1694 mm) in the X (East) direction, +1.1” (+28 mm) in the Y (North) direction, and -18.8” (-478 mm) in the Z (Up) direction. The corresponding numbers for the NE corner are -43.3” (-1100 mm), -2.8” (-71 mm), and -6.6” (-168 mm), respectively. This represents a 3.5% peak lateral tilt in the tower beyond which FRAME3D is unable to follow the structural path to collapse. The other metric of interest in relation to FRAME3D’s performance is the execution time. Shown in Figure 14 is the execution time taken for the incremental dynamic analysis using the Takatori record as a function of the ground motion scaling factor. The analyses were conducted on a high-performance computing cluster, but in an embarrassingly parallel fashion, with each analysis performed on a single core of a Dell PowerEdge 1950 (dual quad core) with a clock-speed of 2.33 GHz and 8 GB RAM. Also shown in the figure is the corresponding analysis time to model instability, beyond which the program is unable to track model collapse. In the FRAME3D analysis of the water-tank,
Figure 10: Time histories of (a) out-of-plane deformations, (b) in-plane deformations, and (c) axial forces in all the braces of Segment 1 of the water-tank under the Takatori ground excitation scaled down by a factor of 0.32. Brace axial forces plotted as a function of brace (d) out-of-plane and (e) in-plane deformations.

Numerical instability occurs only when the tank is physically collapsing. This is apparent from movies of the response of the tank as well as the large one-sided vertical settlement of the tank (Z drift excess of 0.05) that causes the upper structure to tilt and the bulky tank to topple over. To ensure that other cases of numerical instability are not misinterpreted as physical collapse, a 5% threshold may be placed on the Z drift to identify initiation of physical collapse. In addition, animated visualization of model response can help identify collapse initiation by column buckling.
Coordinates of the crests and troughs of all the FRAME3D response histories presented in this paper are catalogued in [12]. An animation of the tank response can be found online [9], while the FRAME3D model can be accessed at the Caltech Virtual Shaker scientific gateway [14]. Nonlinear time-history analysis using FRAME3D can be performed remotely on the model through this gateway. The unscaled Takatori ground motion record is also accessible through the Virtual Shaker’s ground motion database.
Figure 14: Incremental dynamic analysis of the water-tank subjected to scaled Takatori records: (a) Execution time on a single core of a Dell PowerEdge 1950 (dual quad core) with a clock-speed of 2.33 GHz and 8 GB RAM as a function of the ground motion scaling factor. (b) Time from the beginning of ground shaking at which model instability terminates the analysis. Note that there is no numerical instability in the model for ground motion scaling factor below 0.315; the analysis is stable for the entire 40 s duration of the ground motion record.

Model Verification Using PERFORM-3D

To help verify the predictions of the FRAME3D model, a PERFORM-3D [4] model of the water-tank, with features quite similar to the FRAME3D model, is developed. As in FRAME3D, \( P - \Delta \) effects are accounted for in PERFORM-3D by updating nodal coordinates and satisfying dynamic equilibrium in the deformed state in every iteration of a time step. To approximately account for member \( P - \delta \) effects, each structural member in the water-tank is modeled using two elements of equal lengths with two nonlinear fiber segments at the ends and a linear elastic segment sandwiched in between. The two elements combined form a composite fiber element that should effectively emulate the behavior of a single modified elastofiber (MEF) element in FRAME3D consisting of fiber segments at the two ends and in the center, with linear elastic segments sandwiched between either fiber
segment pair (Figure 2). It should be noted, however, that stresses and strains in the fibers in the mid-span segment of the composite element in PERFORM-3D are monitored at two points slightly off center compared to a single point at dead center of the structural member in FRAME3D. An initial imperfection of $5 \times 10^{-6} L$ is specified in either principal direction of all members in both models. As in the FRAME3D model, joints are assumed rigid. Their finite size is accounted for by applying rigid end offsets to the connecting beam elements. The magnitude of these offsets are such that they result in the same effective length for the brace element as in FRAME3D. However, because the brace elements in PERFORM-3D connect to the center of the joint whereas FRAME3D braces connect to the corners of the panel zone element, brace angles are slightly different.

Slabs are modeled using elastic slab/shell elements. All other material, damping, and integration parameters in PERFORM-3D are set to be the same as for the FRAME3D model. The period corresponding to the two orthogonal fundamental translational modes of the PERFORM-3D model is the same as for the FRAME3D model, or 1.31s. The period of the first torsional mode, however, is 0.73s, compared to 0.72s in the FRAME3D model. One important difference between the two programs is that PERFORM-3D adjusts the stiffness matrix contribution to the Rayleigh damping matrix such that there is no axial damping in elements that have one or more fiber segments. This leads to slight differences in the long-time responses of the two models.

The fiber segments of the composite fiber element in the PERFORM-3D model have lengths of 2% of the length of the structural member, the same as that of the fiber segments in the MEF elements in the FRAME3D model. As in FRAME3D, the fiber segments are discretized into 20 fibers that run the full length of the segment. The hysteretic axial stress-strain law associated with each fiber consists of a backbone curve with 5 linear segments culminating in fiber rupture. The hysteretic energy dissipated in successive cycles is controlled by an energy degradation factor $e$ which is the ratio of the area of the degraded hysteresis loop to that of the non-degraded loop. If energy degradation is specified, PERFORM-3D adjusts the unloading and reloading stiffnesses to reduce the area under the hysteresis loop by this factor [4]. The necessary energy degradation can be achieved either by reducing the unloading stiffness or by increasing the hardening stiffness (progressively dropping the yield stress with increasing peak strain under cyclic loading). This behavior is controlled by a user-specified parameter, “the unloading stiffness factor”. Given the differences in the material models of the fiber elements in PERFORM-3D and the MEF element in FRAME3D, it is imperative that the fiber element and the associated material model in PERFORM-3D are properly calibrated and validated prior to use in the water-tank model verification. In the absence of physical test data characterizing the monotonic and cyclic behavior of the water-tank material, PERFORM-3D model calibration and validation are conducted in the following three steps:

(i) **PERFORM-3D material model calibration:**
For this calibration, a stocky (non-buckling) column, fixed at one end and supported on rollers at the other (although, because buckling is precluded, the boundary conditions do not affect the results) is subjected to an axial displacement history [Figure 15(a)]. This history ensures that the fibers traverse through all possible regimes in the stress-strain space. The properties of the column material are identical to that of Strut 19 in the Black et al. cyclic load tests [2], data from which is used for PERFORM-3D validation in the next step. The backbone curve of the axial stress-strain constitutive model of a fiber in PERFORM-3D, the energy degradation relation relating the energy degradation factor $e$ to the peak fiber strain $\epsilon_{\text{max}}$, and the unloading stiffness factor are tuned to achieve a good match between PERFORM-3D and FRAME3D’s predictions of the column fiber axial stress-strain response [Figure 15(b)]. The optimal constitutive model is shown in Figures 15(c) and 15(d). The optimal “unloading stiffness factor” is unity. A factor of unity results in maximum unloading stiffness and minimum elastic range. Fiber material hysteretic rules can be found in [4].

(ii) PERFORM-3D validation:
Figure 16: Validating PERFORM-3D against experimental data (Black et al. [2] strut 19, with KL/r = 40 and one end pinned and the other end fixed, subjected to a gradually growing cyclic displacement history). (a) and (b) Axial force versus axial displacement ($P - \delta$) and axial force versus lateral displacement ($P - \Delta$), respectively, of the strut measured in the test. (c) and (d) Corresponding $P - \delta$ and $P - \Delta$ curves, respectively, from the PERFORM-3D and FRAME3D models. (e) and (f) $P - \delta$ and $P - \Delta$ curves, respectively, from PERFORM-3D and FRAME3D models with fixed-fixed support conditions.
The ability of PERFORM-3D to accurately capture elastic buckling and post-buckling behavior of an element is validated against data from a cyclic load test by Black et al. [2]. The Black et al. testing program comprised 24 steel struts with cross-sectional shapes and slenderness ratios commonly encountered in practice. A36 steel was used for wide-flange and other rolled shapes, and A501 steel was used for square tubes. 18 specimen were pinned at both ends, and had slenderness ratios of 40, 80, and 120, while 6 specimen, with slenderness ratios of 40 and 80, were pinned at one end and fixed at the other. All specimen were subjected to a series of quasi-static, axially applied, displacement and load reversal cycles. Most specimen received a compressive load first, while some were given an initial tensile load. Strut 19 was a wide-flanged section (W6x20), with one end pinned and the other fixed, and a slenderness ratio of 40. Observed mechanical properties included a yield stress of 275.8 MPa (40 ksi) and an ultimate stress of 453 MPa (65.7 ksi). The measured cyclic response of the strut is shown in Figures 16(a) (axial force–axial displacement, $P - \delta$) and 16(b) (axial force–lateral displacement, $P - \Delta$). The corresponding $P - \delta$ and $P - \Delta$ responses, predicted by PERFORM-3D using the calibrated material model from step (i), are shown in Figures 16(c) and 16(d), respectively. Also shown are the responses predicted by a calibrated FRAME3D model for comparison. Details of the FRAME3D model can be found in [11]. The compression and tension capacities of the PERFORM-3D model drop quite rapidly after the first few cycles and the strut fails a few cycles earlier than the experiment. The FRAME3D model, on the other hand, is able to follow the observed response much more closely. Because all elements of the water-tank are assumed continuous, this (analytical) exercise is repeated with both ends assumed fixed. The PERFORM-3D model is able to emulate the FRAME3D model better in this case, although it degrades a couple of cycles earlier than the FRAME3D model [Figures 16(e) and 16(f)].

(iii) Adjusting PERFORM-3D material model for use in the water-tank model:

It is clear that a properly calibrated PERFORM-3D model can follow the buckling and post-buckling response of slender members quite effectively. For application to the water-tank problem, however, the material model needs to be adjusted to reflect the differences in the properties of steel used in that problem and that used for Black et al.’s strut 19. To this end, the exercise in step (i) is repeated for a non-buckling stocky column with material properties corresponding to that of tank steel. The optimal backbone curve describing the fiber axial stress-strain behavior and the optimal $e - \epsilon_{max}$ relationship are shown in Figures 17(a) and 17(b), respectively. As in the case of the optimal material model for Black et al.’s strut 19, the optimal unloading stiffness factor is unity. The corresponding comparison of PERFORM-3D response against FRAME3D response is shown in Figure 17(c).
Figure 17: Adjusting the calibrated PERFORM-3D material model for the material properties of steel used in the water-tank. Comparison is made against FRAME3D’s prediction of the hysteretic behavior of a stocky (non-buckling) column. (a) Optimal backbone curve for the axial stress-strain behavior of fibers in PERFORM-3D overlaid on the corresponding FRAME3D curve. (b) Optimal $e - \epsilon_{\text{max}}$ relationship in PERFORM-3D for water tank material. (c) PERFORM-3D and FRAME3D plots of fiber axial stress versus strain (all fibers exhibit the same stress-strain response).

Comparison of FRAME3D and PERFORM-3D models of the water-tank

Three sets of analyses are conducted to compare the PERFORM-3D and FRAME3D models of the water-tank:

(i) Pushover analysis:

The water-tank models are subjected to a slow, ramped, horizontal ground acceleration that increases at a constant rate of 0.3 g/minute, and their dynamic (effectively pseudo-static) responses are computed using PERFORM-3D and FRAME3D. The pushover curves (base shear as a function of the roof displacement) are shown in Figure 18(a). The peak shears are nearly identical. Collapse is triggered at a slightly smaller roof displacement in the PERFORM-3D model. Furthermore, columns in the PERFORM-3D model lose their axial force carrying capacity much more rapidly (at smaller axial and lateral displacements) than in the FRAME3D model. This can be seen in Figures 18(c) and 18(d). The more gradual loss of axial force carrying capacity (relative to the axial and lateral displacements) in the southwest column of Segment 1 of the FRAME3D model
Figure 18: Response time histories of the FRAME3D and PERFORM-3D models of the water-tank in a pushover analysis: (a) Displacements of the north-east corner of the roof plotted against base shear. (b) Axial forces, and (c) $\delta$ and (d) $\Delta$ displacements of the SW column in Segment 1.

(ii) Dynamic analysis using the Takatori record:

The PERFORM-3D model of the water-tank is analyzed under the 3-component Takatori record, scaled to various levels. It is found that the smallest scaling factor at which the PERFORM-3D model collapses is 0.323 (compared to 0.315 for the FRAME3D model). Shown in Figure 19 are the comparisons of the PERFORM-3D water-tank roof response [northeast corner with initial coordinates $(X, Y, Z) = (160, 320, 1920)$] at the
Table 1: Fiber rupture “events” that cause sudden drops in the axial load carrying capacity of the SW column in Segment 1 of the FRAME3D water-tank model during a pushover analysis. The event numbers are labeled on the column force-displacement curve shown in Figures 18(c) and 18(d).

<table>
<thead>
<tr>
<th>Event No.</th>
<th>Segment ID</th>
<th>Fibers that Rupture</th>
<th>Time (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>7-12, 16, 20</td>
<td>151.485</td>
</tr>
<tr>
<td>2a</td>
<td>2</td>
<td>19</td>
<td>151.520</td>
</tr>
<tr>
<td>2b</td>
<td>2</td>
<td>15</td>
<td>151.525</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>1-6</td>
<td>151.710</td>
</tr>
<tr>
<td>4a</td>
<td>2</td>
<td>18</td>
<td>151.715</td>
</tr>
<tr>
<td>4b</td>
<td>2</td>
<td>14</td>
<td>151.725</td>
</tr>
<tr>
<td>5a</td>
<td>1</td>
<td>13</td>
<td>151.730</td>
</tr>
<tr>
<td>5b</td>
<td>1</td>
<td>17</td>
<td>151.745</td>
</tr>
<tr>
<td>6</td>
<td>1</td>
<td>14</td>
<td>151.880</td>
</tr>
<tr>
<td>7</td>
<td>1</td>
<td>18</td>
<td>151.915</td>
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<tr>
<td>8a</td>
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<td>5,6</td>
<td>151.940</td>
</tr>
<tr>
<td>8b</td>
<td>3</td>
<td>1-4</td>
<td>151.945</td>
</tr>
<tr>
<td>9a</td>
<td>3</td>
<td>17</td>
<td>151.955</td>
</tr>
<tr>
<td>9b</td>
<td>3</td>
<td>13</td>
<td>151.985</td>
</tr>
<tr>
<td>10</td>
<td>3</td>
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<td>152.090</td>
</tr>
<tr>
<td>12</td>
<td>3</td>
<td>14</td>
<td>152.170</td>
</tr>
</tbody>
</table>

Figure 19: Roof displacement time histories of the PERFORM-3D model under the Takatori record scaled down by factors of 0.32 and 0.323, and that of the FRAME3D model at the benchmarking scaling level of 0.32.

benchmarking scaling level of 0.32 as well as at the collapse threshold level of 0.323 against that of the FRAME3D model at the benchmarking scaling level. Responses in all three cases match quite well until after the strongest near-source pulse (Figure 5) has impacted the tank (~6.7 s). At this stage, the Segment 1
columns on the west face have buckled [Figures 20(a), 20(c), 21(a), 21(c), 22(a), and 22(c)]. The post-buckling response of these columns as modeled by PERFORM-3D is different from that modeled by FRAME3D. First, the FRAME3D columns experience greater lateral deformation. Furthermore as shaking progresses, the axial and lateral deformation in the southwest column of the FRAME3D model gradually increase, while that in the northwest column holds steady [Figures 21(a), 21(c), 22(a), and 22(c)]. This causes the period of oscillation of the FRAME3D model to lengthen (Figure 19). No corresponding progressive stiffness degradation is observed in the PERFORM-3D columns; the column axial and lateral deformations hold steady after the first instance of buckling. No significant period-lengthening is observed in the PERFORM-3D model (Figure 19).

Figure 20: Time histories of axial forces in columns in Segment 1 in the FRAME3D model of the water-tank under the Takatori record scaled down by a factor of 0.32 and the PERFORM-3D model under the Takatori earthquake scaled down by factors of 0.32 and 0.323.
Figure 21: Time histories of $\delta$ in columns in Segment 1 in the FRAME3D model of the water-tank under the Takatori record scaled down by a factor of 0.32 and the PERFORM-3D model under the Takatori earthquake scaled down by factors of 0.32 and 0.323.

As shaking progresses, fibers in the Segment 1 columns on the west face of the FRAME3D model start rupturing, leading to a finite reduction in strength each time a fiber ruptures, and a more gradual loss in column strength. Columns in the PERFORM-3D model, on the other hand, lose their strength more suddenly (Figure 20). It may be recalled that the sudden loss of strength in the columns of the PERFORM-3D model was also observed in the pushover analysis of the water-tank [Figure 18(b)]. The southwest and northwest columns in Segment 1 of the PERFORM-3D model completely lose their axial load-carrying capacity (Figure 20) at 13.69 s and 13.74 s, respectively, closely followed by the northeast and southeast columns at 14.33 s and 14.50 s, respectively. With the complete loss of all columns in Segment 1, the tank collapses in a north-westerly
Figure 22: Time histories of $\Delta$ in columns in Segment 1 in the FRAME3D model of the water-tank under the Takatori record scaled down by a factor of 0.32 and the PERFORM-3D model under the Takatori earthquake scaled down by factors of 0.32 and 0.323.

direction (Figure 19). The Segment 1 columns on the east face of the FRAME3D model, however, do not lose their axial load carrying capacity, and the tank collapses in a westerly direction. Thus, while the initiation of collapse in the FRAME3D and PERFORM-3D models is identical, the eventual collapse footprints are a little different. Snapshots of the deformed shape of the PERFORM-3D model (exaggerated by a factor of 5) under the Takatori record scaled down by a factor of 0.323 are shown in Figure 23. They are quite similar to those of the FRAME3D model (Figure 8).

The degree of agreement between the FRAME3D and PERFORM-3D models can be gauged by computing the coefficient of correlation $\rho$ between 4 s windows of FRAME3D and PERFORM-3D model response histories.
Figure 23: (a) 7 s, (b) 13.65 s, and (c) 13.83 s snapshots of the collapsing PERFORM-3D model of the benchmark steel braced water-tank subjected to the 1995 Kobe earthquake Takatori record scaled down by a factor of 0.323. Deformations are exaggerated by a factor of 5.

Figure 24: Evolution of 4 s moving window correlation coefficient $\rho$ between FRAME3D and PERFORM-3D roof corner E, N, and Z displacement histories under the Takatori record scaled by a factor 0.32. Also shown are the windowed correlation coefficients between these histories and the PERFORM-3D roof displacement response histories under the Takatori record scaled by a factor 0.323. The gray area, representing correlation coefficients higher than 0.9, is indicative of excellent agreement between the corresponding responses.

The time evolution of $\rho$ can be tracked by shifting the windows by, say, 1 s steps. Figure 24 shows the 4 s moving window correlation coefficients between the FRAME3D and PERFORM-3D model roof east, north, and vertical displacement histories at a corner node [with initial coordinates $(X, Y, Z) = (160, 320, 1920)$]. Excellent agreement ($\rho > 0.9$) is obtained for the first 9 s of the records. By this time, of course, columns have already buckled and the FRAME3D model has started to degrade and has embarked on its gradual descent to collapse. The PERFORM-3D model does not degrade in this gradual manner, but exhibits sudden instability at around 14.4 s. Thus, the correlation coefficient between FRAME3D and PERFORM-3D responses drops
significantly between 11 s and 14 s, but gets back up to about 0.65 toward the end of the record.

(iii) Dynamic analysis using 17 other near-source records:

To further verify the FRAME3D model against the PERFORM-3D model under a broader range of ground motions, both models are analyzed under 17 near-source records from the 1992 Cape Mendocino, the 1999 Chi-Chi, the 1979 Imperial Valley, the 1978 Iran, the 1995 Kobe, the 1989 Loma Prieta, the 1992 Landers, the 1994 Northridge, the 1971 San Fernando, and the 1987 Superstition Hills earthquakes, scaled by a factor of 0.75. The 5% damped pseudo-acceleration response spectra of these records are shown in Figure 25.

Figure 25: 5% damped pseudo-acceleration response spectra of the strong component of 17 near-source ground motion records (unscaled) used for FRAME3D-PERFORM-3D model response comparison. The records are from the 1992 Cape Mendocino, the 1999 Chi-Chi, the 1979 Imperial Valley, the 1978 Iran, the 1995 Kobe, the 1989 Loma Prieta, the 1992 Landers, the 1994 Northridge, the 1971 San Fernando, and the 1987 Superstition Hills earthquakes.

The peak roof displacements in the X, Y, and Z directions of both models are listed in Table 2. Of the 17 cases, the FRAME3D model collapses in 3 cases, whereas the PERFORM-3D model collapses in 4 cases. The only anomalous case is that of the Tabas record, where the FRAME3D model just manages to survive. Solid lines in Figure 25 represent the four cases where the PERFORM-3D model collapses, while dashed lines correspond to cases where the model does not collapse. In the cases with model collapse, the peak roof displacement at numerical instability of the FRAME3D model is much smaller than that in the PERFORM-3D model, suggesting that PERFORM-3D is able to track the progress of collapse in the structure farther along when compared to the FRAME3D model. However, the external work-internal energy balance is significantly disrupted as the model collapses, i.e., equilibrium errors could be significant and the results may not be trustworthy [4]. For example, the maximum energy imbalance in the PERFORM-3D analysis in two cases where collapse does not occur – the Pacoima Dam record from the San Fernando earthquake and the Lexington Dam record from the Loma Prieta earthquake – are only 0.002% and 0.173%, respectively. In comparison, the
work-energy imbalance jumps to 39.3% for the Tabas record (Iran earthquake), 20.4% for the Rinaldi record (Northridge earthquake), 32.0% for the Superstition Hills record (Superstition earthquake), and 174.0% for the Sylmar record (Northridge earthquake), cases where the PERFORM-3D model collapses – a rather steep drop in model veracity. As evident from the peak Z drift in Segment 1 and Segments 2-5 listed in the table, collapse in both the FRAME3D and the PERFORM-3D models is always initiated in Segment 1 (due to asymmetric column buckling), followed by rigid body toppling of the upper segments and the tank. In the 13 cases where neither model collapses, the peak roof displacement responses from the FRAME3D model agree very well with those from the PERFORM-3D model.

<table>
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<tr>
<th>Ground Motion Record</th>
<th>Maximum Absolute Roof Displacement (m)</th>
<th>Collapse?</th>
<th>Peak Z Drift</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FRAME3D (F3D)</td>
<td>PERFORM-3D (P3D)</td>
<td>F3D</td>
</tr>
<tr>
<td>Cape Mendocino</td>
<td>0.21 0.12 0.08</td>
<td>0.21 0.12 0.09</td>
<td>No</td>
</tr>
<tr>
<td>Chi-Chi, CWBC101</td>
<td>0.17 0.16 0.07</td>
<td>0.19 0.17 0.07</td>
<td>No</td>
</tr>
<tr>
<td>Chi-Chi, CWBT063</td>
<td>0.14 0.20 0.09</td>
<td>0.17 0.22 0.10</td>
<td>No</td>
</tr>
<tr>
<td>Imperial Valley, El Centro #6</td>
<td>0.12 0.09 0.05</td>
<td>0.14 0.09 0.07</td>
<td>No</td>
</tr>
<tr>
<td>Imperial Valley, El Centro #7</td>
<td>0.17 0.15 0.07</td>
<td>0.17 0.17 0.09</td>
<td>No</td>
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<tr>
<td>Kobe, JMA-Kobe</td>
<td>0.28 0.21 0.08</td>
<td>0.29 0.21 0.09</td>
<td>No</td>
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<tr>
<td>Loma Prieta, Lexington Dam</td>
<td>0.24 0.18 0.12</td>
<td>0.26 0.19 0.12</td>
<td>No</td>
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<tr>
<td>Loma Prieta, Los Gatos</td>
<td>0.36 0.07 0.08</td>
<td>0.37 0.07 0.09</td>
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<tr>
<td>Loma Prieta, Los Gatos</td>
<td>0.27 0.17 0.12</td>
<td>0.31 0.16 0.13</td>
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<td>Landers, Lucern Valley</td>
<td>0.27 0.08 0.09</td>
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<td>Imperial Valley, Meloland</td>
<td>0.22 0.15 0.06</td>
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<td>No</td>
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<tr>
<td>Northridge, Pacoima Dam</td>
<td>0.28 0.13 0.13</td>
<td>0.29 0.16 0.16</td>
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<tr>
<td>San Fernando, Pacoima Dam</td>
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<td>Northridge, Rinaldi</td>
<td>1.32 0.21 0.17</td>
<td>8.28 0.52 3.66</td>
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<td>Superstition, Superstition Mt.</td>
<td>1.23 0.30 0.27</td>
<td>7.58 0.28 3.55</td>
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<td>Northridge, Sylmar</td>
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<td>4.25 0.66 7.13</td>
<td>Yes</td>
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<tr>
<td>Iran, Tabas</td>
<td>0.28 0.33 0.19</td>
<td>0.48 1.85 5.77</td>
<td>No</td>
</tr>
</tbody>
</table>

**Table 2**: Comparison of FRAME3D and PERFORM-3D model responses under 17 scaled near-source records.
Figure 26: Peak absolute roof corner E, N, and Z displacement in 4s moving windows in the (a)-(c) FRAME3D and the (d)-(f) PERFORM-3D models under 17 near-source records. (g)-(i) Evolution of 4 s moving window correlation coefficient $\rho$ between the FRAME3D and the PERFORM-3D roof corner E, N, and Z displacement histories. The gray area, representing correlation coefficients higher than 0.9, is indicative of excellent agreement between the corresponding responses.
As for the scaled Takatori record, the time evolution of the correlation coefficient between 4 s windows of the FRAME3D and the PERFORM-3D model roof corner node [with initial coordinates \((X, Y, Z) = (160, 320, 1920)\)] east, north, and vertical displacement histories is shown in Figure 26. Once again, solid lines represent the four cases where the PERFORM-3D model collapses, while dashed lines correspond to cases where the model does not collapse. The lateral diaplacement correlation coefficient rarely drops below 0.9 for the 13 cases where collapse does not occur in both models. Even in the 3 cases where both models collapse, \(\rho\) is greater than 0.9 for the most part, except for the last one or two time windows. Also shown on the figure are the peak responses in the FRAME3D and the PERFORM-3D models over the same time windows. The agreement between the lateral displacement envelopes in the cases with no collapse are excellent, whereas the agreement between the corresponding vertical displacement envelopes is only marginally poorer.

**Conclusions and Discussion**

A case study of the collapse of a water-tank under earthquake loading is presented as a potential candidate for low-complexity benchmarking in the evaluation of the collapse-prediction capability of competing methodologies. The tank is so configured as to have a mechanistically inferable characteristic collapse mechanism when excited by ground motion. It is biaxially symmetric and is supported by a steel tower that consists of four sloping mega-columns that are tied together by five levels of vertical and horizontal bracing. The braces, columns, and beams have uniform sizing for the entire height of the tower. This configuration forces the collapse of the tank to occur due to buckling of the mega-columns and braces in the bottom segment of the steel lattice, and overturning due to ensuing \(P - \Delta\) instability. Incremental dynamic analysis is conducted on a FRAME3D model of the tank, subjected to the Takatori near-source record from the 1995 Kobe earthquake. MEF elements are used to model all the members including braces. Collapse is found to occur at the threshold scaling factor of 0.315. Severe buckling occurs in the bottom mega-columns and one of the two braces on the west face of the tower when the structure is impacted upon by the Takatori near-source pulse, resulting a tilt in the structure. This is followed by sequential compression buckling of braces on the south and north faces leading to \(P - \Delta\) instability and complete collapse of the tank. The results of the FRAME3D model compare very well against those from a calibrated and validated PERFORM-3D model. The sequence of member buckling and the triggered collapse mechanism are identical. Differences include the eventual collapse footprint and the collapse threshold ground motion scaling factor (0.323 for the PERFORM-3D model). For collapse benchmarking, an analysis of the water-tank subjected to the Takatori ground motion record, scaled by a factor of 0.32, is suggested. Time histories from this analysis case using FRAME3D are catalogued in a Caltech EERL technical report available online. The model itself can be accessed online at the Caltech Virtual Shaker. It is the authors’ expectation that
other research groups would repeat the case study using software such as OpenSEES, LS-DYNA, ABAQUS, etc., and develop the problem further. It is hoped that these efforts culminate in a series of well-documented benchmark problems of increasing complexity for use by the collapse-modeling community at large, similar to the series developed by the structural health monitoring and control community.

To judge the adequacy of a model, we propose the establishment of validation metrics to quantify the degree of agreement with the benchmark model on the collapse mechanism, the onset of buckling in the various members leading to global collapse, the post-buckling behavior of columns and braces (hysteretic loops), period-lengthening observed in horizontal force resultants, the tank roof displacement and trajectory as it collapses, time beyond which the algorithm is unable to track the progression of collapse, etc. Time histories could be compared on the basis of correlation coefficients computed either over the entire length of the records or over a sequence of moving windows as adopted here. Models could be deemed adequate if they satisfy certain pre-determined thresholds on these metrics. For all the quantitative metrics, a common threshold of, say, within ±20% of benchmark value could be adopted.
REFERENCES


