Hybrid Simulation for System-Level Structural Response

Justin Adam Murray¹, Mehrdad Sasani², and Xiaoyun Shao³

Abstract

Hybrid simulations combine physical and analytical components into a single simulation to evaluate the response of a structure, often under seismic ground motion. This allows an experiment to be conducted in which structural components with complex response can be modeled experimentally and more well-known components can be represented within an analytical model. The coordination software UI-SimCor, developed by the MUST-SIM NEES facility at the University of Illinois at Urbana-Champaign, is a hybrid simulation tool which performs the dynamic analysis and other software and hardware coordination tasks for hybrid simulations. In many hybrid simulations, including those that have used UI-SimCor, analytical models with few effective degrees of freedom are typically used. In simulations where system-level behaviors and the response of the analytical components are of importance, a more detailed analytical system is needed. This changeover to a more complex analytical system and increase in general complexity of the hybrid simulation can cause various issues within the UI-SimCor framework. This study discusses the difficulties and issues that arise from having large and complex analytical substructures in hybrid simulation, and the effective mitigation or solutions to those problems.

Keywords: Hybrid Simulation, Reinforced Concrete, Large-Scale Testing, OpenSees, Structural Failure

¹ PhD Candidate, Department of Civil and Environmental Engineering, Northeastern University, Boston, MA, Email: murray.jus@husky.neu.edu

² Associate Professor, Department of Civil and Environmental Engineering, Northeastern University, Boston, MA Email: sasani@neu.edu, Tel: 617-373-5222, Fax: 617-373-2444

³ Associate Professor, Department of Civil and Construction Engineering, Western Michigan University, Kalamazoo, MI, Email: xiaoyun.shao@wmich.edu

Introduction

Hybrid simulations (HS) combine physical and analytical components (substructures) into a single simulation to obtain a system-level structural response under dynamic ground motion (GM). HS allows for evaluation of entire building systems, with the potential for experimental testing of specific structural components at large scale so that both experimental and analytical behaviors and the interaction between them can be observed. One common practice of HS is that complex mechanisms or components are modeled experimentally (and often at large-scale) while more well-known components are represented within an analytical model.

This study focuses on the development of a HS approach to help achieve the final goal of a Network for Earthquake Engineering Simulation Research (NEESR) project, which is to evaluate system level response of existing reinforced concrete (RC) structures following shear-axial column failure. Although the findings and observations presented herein are applicable in many HS procedures, this study and the presented advancements are related to the use of the UI-SimCor coordination software at the University of Illinois (Kwon et al., 2005, Kwon et al., 2008). Other similar architectures for HS would similarly benefit from the advancements presented in this study.

Shear-axial column failure, and the system-level collapse resistance of a building after such a failure, is an important topic in RC retrofitting and seismic analysis, as many buildings built prior to the mid-1970s have column detailing which make them susceptible to shear-axial failure. Shear-axial column failure is a difficult mechanism to predict, and current failure models and capacity equations include a high degree of uncertainty and variability (Lynn et al., 1996; Elwood, 2004; Sasani 2007; Henkhaus et al., 2013). However, the continuous development of HS

makes it feasible to analyze a building's response to such a column failure and to understand the collapse-resisting mechanisms of the structure under such extreme conditions.

Modern HS originated from the pseudodynamic testing method firstly reported by Takanashi et al. (1975). Further research on the use of the pseudodynamic method for seismic testing, such as quantifying experimental errors, limitations, and stability issues, was conducted by Mahin and Shing (1985). HS methods have since underwent an accelerated development phase reflected by increasingly complex analytical and physical substructures, employment of high-performance hydraulic loading equipment and development of various HS coordination software (Elnashai et al. 2004, Shing et al., 2007). HS have also been conducted with geographic distribution of test hardware and software. Yang et al. (2009) performed a geographically distributed HS, testing zipper frames with components of the test located at both the University of California at Berkeley and at University of Colorado at Boulder. The displacement-based pseudodynamic HS method prevails in recent implementations (Shao and Griffith, 2013) due to its similar testing setup to a quasi-static test and the many available supporting open-source programs, such as OpenFresco (Schellenberg et al., 2007) and UI-SimCor (Kwon et al., 2005, Kwon et al., 2008).

Hashemiet al. (2014) discuss the challenges regarding convergence and spurious oscillation that can occur when conducting HS with large numerical substructures. One way to address the challenge of complex numerical modeling in HS is to develop HS coordinating systems that utilize the dynamic solution algorithms within existing FEM software. For example Wang et al. (2008) developed a P2P HS system that combines physical testing with numerical substructures modeled in OpenSees and ABAQUS. The architecture of the P2P system allows the equations of motion to be formulated and solved within each substructure's domain using their respective software and only maintaining the displacement and force balances at the boundaries using an

initiative algorithm. Another example is OpenFresco (Schellenberg et al., 2010) which serves as a middleware between physical testing and a numerical simulation conducted in OpenSees. Thus OpenSees is utilized not only for numerical modeling, but also for the dynamic solution of the entire structure model being investigated. Further advancement in OpenFresco were made recently by Hashemi et al. (2014) who proposed an innovative overlapping substructuring technique for collapse HS of steel frame when large deformations were anticipated.

On the other hand, UI-SimCor adopted a different HS coordinator architecture, and is the primary coordinator used at the MUST-SIM lab at the University of Illinois at Urbana-Champaign (UIUC). So far UI-SimCor has been successfully implemented in large scale HS of RC bridges of various configurations (Spencer et al. 2006, Kim et al., 2011, Abdelnaby et al., 2014) and semi-rigid steel frames (Mahmoud and Elnashai, 2013). It has also been used for small scale HS of 6-story steel frames (Lin, 2010) and RC bridges (Li et al., 2013). While some of these experiments have complex numerical substructures, they had a limited number of *effective DOFs* (discussed in the following section).

In this study, a 10-story RC building is analyzed under a severe earthquake GM, and is substructured for HS to allow physical experimentation of three lower-floor columns in the MUST-SIM NEES facility at UIUC, with the remainder of the building system modeled in the computer program OpenSees (McKenna et al., 2002). A large three-dimensional analytical substructure (thousands of DOFs) is used with nonlinear modeling of all elements, as collapse-resistance mechanisms in building structures often involve large amounts of inelastic deformation. The challenges that arise from using such a complex numerical model and potential adjustments to the HS coordinator/architecture in order to mitigate these challenges is presented in the following sections.

Prototype building and analytical model

In this section, the analytical model that is used in the HS is presented. Note again that in this paper the main objective is to develop a HS method that is applicable for evaluating the response of such a model and to demonstrate the issues and the proposed solutions. To conduct a successful HS, which will allow for evaluation of system-level behaviors of older RC buildings subjected to seismic GM, a representative pre-1970s 10-story RC building structure is designed according to ACI 318-63 (1963), and a detailed analytical model of the structure is developed. The building is designed based on a review of the design and layout of several actual RC structures constructed during that time period (John A. Blume & Associates, 1973). Typical building characteristics included tall 1st stories (for lobbies, etc.), as well as deep spandrel beams with shallower interior beams, allowing for more interior space and higher ceilings, while putting more of the seismic demand on the exterior of the structure. More discussion of the review of pre-1970s construction can be found in Murray and Sasani (2013). For the representative structure, the building plan consists of two 6.10m (20') spans in the north-south (transverse) direction and six 6.10m(20') spans in the east-west (longitudinal) direction. The structure has 400mmx710mm (16"x28") spandrel beams on the first floor with 400mmx560mm (16"x22") spandrel beams on other floors. Interior beams are 400mmx460mm (16"x18") on all floors. The first story height is 4.27m (14'-0"), while other stories have a height of 3.20m (10'-6"). Floors consist of one-way 115mm (4.5") slab, supported by intermediate beams at the midpoint of longitudinal beams. Concrete is designed with an $f'_c=27.6$ MPa (4 ksi) and all steel is Grade 60. A typical floor plan is shown in Figure 1. Note that the second floor is identified as the top of the first story.

The representative building is modeled in OpenSees (McKenna et al., 2002), an open source modeling software capable of nonlinear modeling of building structures. Two-node beam-column elements are used to model all beams, columns, and slabs within the structure. Elements are nonlinear using a concrete constitutive relationship with no tensile strength, and a bilinear steel model with 2% strain hardening. A force-based formulation is used for the beam elements, which compared to a displacement formulation, allows for a reduced number of nodes and elements in the model. Columns of the structure have 4 integration points while beams and slabs have 3 integration points (but are represented with more elements). The slabs of the floor system are not modeled as shells, but rather as a grid of nonlinear beam elements (sized based on their tributary areas), using half of their linear torsional stiffness (MacLeod, 1990). Nonlinear shell elements are not used because they have not been extensively verified in OpenSees, and the use of a grid of beam elements representing slabs has been successfully used in the past with good agreement with physical data (e.g. Sasani et al., 2011). Beam sections are modeled as T and L sections to account for the contribution of the adjacent slab. The layout of nodes and connecting elements in the structural model in OpenSees are shown for a typical floor plan in Figure 2. Note that the node clusters around columns are used for modeling the rigid joints of the system and the bar-slip deformations at the faces of the joints. Joint damage is outside the scope of this study so the joints are modeled using stiff linear elements. Bar-slip deformations were accounted for through modification of the Gauss-Lobatto integration point weights to amplify deformations at the element ends. The amount of amplification was calibrated against column tests by Sezen (2002) and Lynn et al. (1996).

The gravity loads of the structure are distributed among all of the nodes in the floor based on their tributary area. The live load is assumed to be 3.11 kN/m^2 (65psf) for design, which is used

as an average value representing the live load applied to different parts of the floors. The floors are also subjected to 1.92 kN/m^2 (40psf) of dead load in addition to the weight of the structural components, accounting for partitions, floor finishing, drop ceilings, as well as mechanical, piping and fixtures. Masses used in the seismic analysis are distributed among all of the nodes, similar to the distribution of the gravity forces, and include the dead load and 25% of the live load (GSA, 2003). Bar-slip behavior is explicitly accounted for in the OpenSees model. Bar-slip is a flexural response that occurs in RC elements which results in concentrated rotations at the end of beams and columns due to strain penetration of the rebar into joints and foundations. The model used in this study was calibrated against several large-scale tests which measured the effect of these rotations on the response (Lynn et al., 1996; Sezen, 2002). As the purpose of this paper is to discuss the HS process as it relates to a large analytical model and not the specifics of modeling techniques in OpenSees, further discussion of the details of the analytical model are not presented here. For more info regarding analytical modeling techniques used in this project, see (Murray and Sasani 2013 and 2014).

Overview of Statically-Condensed Hybrid Simulation (SC-HS)

Within pseudodynamic HS, there are several methods used to analyze the structural system as a whole, generally through a centralized *coordinator*. Coordinators typically handle various tasks within a HS, including communication between substructures, solution of the equation of motion, and data logging. OpenFresco and UI-SimCor remain the most popular tools for coordinating a HS in the US.

A static condensation can be used to eliminate from dynamic analysis those degrees of freedom (DOFs) of a structure to which zero mass is assigned (Chopra, 2012). In a HS due to the

nature of the procedure, one usually includes not only DOFs that have mass, but also the ones which have gravity (or other static) loads applied as well as the DOFs at the boundaries between different substructures. These DOFs are collectively called “effective” DOFs in this paper. HS using such static condensation will from this point forward is defined as a statically-condensed hybrid simulation (SC-HS). SC-HS can be described using the following partitioned differential equation, which represents the equilibrium of all DOFs of the structure.

$$\begin{bmatrix} m_e & 0 \\ 0 & 0 \end{bmatrix} \begin{Bmatrix} \ddot{u}_e \\ \ddot{u}_s \end{Bmatrix} + \begin{bmatrix} k_{ee} & k_{es} \\ k_{es}^T & k_{ss} \end{bmatrix} \begin{Bmatrix} u_e \\ u_s \end{Bmatrix} = \begin{Bmatrix} P_e \\ P_s \end{Bmatrix} \quad (1)$$

where $\ddot{u}$, and u are the acceleration and displacement vectors of the system. For clarity of the discussion that follows, the damping term(a function of the velocity vector $\dot{u}$) is excluded from the equation and will be discussed later. The subscript e represents the effective DOFs whose response will be evaluated solving the dynamic equations of motion. The subscript s represents the remaining DOFs of the structure, whose response will be evaluated by either solving static equilibrium equations or experimentally. m_e is the lumped mass matrix associated with the effective DOFs, which has some zero diagonal terms associated with the DOFs which either are at the boundaries of the substructures or have gravity loads, but no mass. The k submatrices in the second term in (1) represent the portions of the stiffness matrix of the structure associated with the effective and the remaining DOFs as well as the corresponding cross terms. Based on the definition of the effective DOFs presented in the previous paragraph, all the components of the P_s vector are zero. This is due to the fact that P_e includes all static and dynamic forces applied to the structure. But for a more general definition of effective DOFs, which will be discussed later, P_s is not replaced with a zero vector yet.

The partitioned equations in (1) are

$$m_e \ddot{u}_e + k_{ee} u_e + k_{es} u_s = P_e \quad (2a)$$

$$k_{ss} u_s = P_s - k_{es}^T u_e \quad (2b)$$

From (2b) u_s can be found as

$$u_s = (k_{ss})^{-1} P_s - (k_{ss})^{-1} k_{es}^T u_e \quad (3)$$

and replacing u_s from (3) in (2a), we arrive at

$$m_e \ddot{u}_e + k_{ee}^* u_e = P_e - k_{es} (k_{ss})^{-1} P_s \quad (4a)$$

where k_{ee}^* is the statically condensed stiffness matrix of the structure.

$$k_{ee}^* = k_{ee} + k_{es} (k_{ss})^{-1} k_{es}^T \quad (4b)$$

In (4a) if P_s is set equal to zero ($P_s \neq 0$ will be discussed later) we will have

$$m_e \ddot{u}_e + k_{ee}^* u_e = P_e \quad (5)$$

Note that Rayleigh damping can be readily accounted for in (5) using a damping matrix as a linear combination of m_e and k_{ee}^* .

$$m_e \ddot{u}_e + c_{ee}^* \dot{u}_e + k_{ee}^* u_e = P_e \quad (6)$$

In a linear structure, u_e and then u_s can be found from (6) and (3), respectively. Under a general loading condition and particularly for a nonlinear structure, (6) is solved incrementally.

$$m_e \Delta \ddot{u}_e + c_{ee}^* \Delta \dot{u}_e + k_{ee}^* \Delta u_e = \Delta P_e \quad (7)$$

The dynamic analysis in SC-HS can be conducted using a non-iterative time-integration scheme. The operator splitting method in conjunction with the α -modified Newmark scheme (α -OS method) has been implemented as the integration algorithm in UI-SimCor. The α -OS

197 integration scheme was proposed by Combescure and Pegon (1997). Compared to an iterative
 198 implicit integration scheme for pure numerical simulation, the α -OS method does not require
 199 iterations within each time step to achieve equilibrium for nonlinear structural responses, which
 200 is a desired feature for HS purposes, as perturbing or iterative motion of a physical specimen
 201 can be avoided. The α -OS method uses the initial stiffness of all substructures to solve (7), that is,
 202 it replaces k_{ee}^* with the statically condensed initial stiffness k_l .

203 Due to the nonlinearity of the system, the solution to (7) needs to be corrected. For this
 204 purpose, the solution (incremental displacements at effective DOFs) is sent as commands to be
 205 executed by each analytical and experimental substructure to obtain the resisting forces. To
 206 evaluate the status of total equilibrium in nonlinear systems, $k_{ee}^* u_e$ in (6) must be replaced by the
 207 resisting force vector $p_r^*(u_e)$.

$$208 \quad m_e \ddot{u}_e + c_{ee}^* \dot{u}_e + p_r^*(u_e) = P_e \quad (8)$$

209 Within a SC-HS, all substructures, regardless of their size or whether they are experimental or
 210 analytical, are treated equally and are collectively referred to as substructures or ‘modules’. The
 211 experimental substructures find their resisting force in the laboratory. The analytical substructure
 212 of a HS would need to apply the displacement commands through some form of constraint
 213 problem. A flow chart showing the sequence of steps in a SC-HS performed in UI-SimCor is
 214 shown in Figure 3, as it pertains to this project. During initialization, information regarding the
 215 model and analysis, such as the mass, damping, and initial stiffness matrices, are defined.
 216 Stiffness matrices are defined for each module individually and assembled into a global stiffness
 217 matrix by the program.

Note that as shown in Figure 3, simulation coordinators handle several aspects of a SC-HS. They perform the dynamic time-integration solved under the defined loading or GM. They also typically handle the communication between various substructures of the simulation, passing displacements commands onto physical specimens and analytical models as well as collecting the restoring forces and assembling the restoring force vector. Some coordinators, such as UI-SimCor, have additional capabilities in terms of graphical user interfaces for tracking data in real time, triggering data acquisition (DAQ) systems, as well as automatic control of cameras in the laboratory.

SC-HS has typically been used to analyze systems with few effective DOFs to minimize computational effort within the coordinator, since the coordination software solves both static (for gravity) and dynamic equilibrium equations, in addition to performing various other tasks. Sometimes the complexity amounts to as little as 1 or 2 effective DOFs. Note that this does not inherently limit the complexity of the analytical modules, which can have orders of magnitude more DOFs than the model analyzed by the coordinator, and are solved within a structural analysis software. Due to the required distribution of masses and loads (for near-collapse analysis of a 3D structure) in this study, was a need to drastically increase the number of effective degrees of freedom in the HS beyond what has been done in UI-SimCor in the past. Note again that other tests have included many DOFs, but typically have had few *effective DOFs* (masses, loads, boundary locations). This paper will discuss the process and difficulties of doing so.

Issues with SC-HS for Large Analytical Systems

Oscillations in Higher-Frequency Modes and Damping in alpha-OS

Previous HSs have noted that the higher-mode response of a system can be erroneously excited causing spurious oscillations between time steps (Nakashima and Kato 1988; Combescure and Pegon, 1997). Combescure and Pegon proposed the use of alpha-factor numerical damping (alpha-OS method) to dissipate these higher-mode contributions from the results. However, in our analyses of the 10-story RC structure with a large number of effective DOFs, the alpha-damping did not mitigate the extreme oscillations that can occur in the simulation.

To illustrate the oscillations that can occur, an analytical example was conducted, in which the 10-story system is represented by an OpenSees model acting as the only module (substructure) communicating with UI-SimCor. There are a total of 470 effective DOFs in the model (470 masses and static loading points). The analysis consists of 100 steps of gravity loading, followed by 100 steps of stabilization, in which no additional loads are applied. Shortly after the start of the dynamic portion of the analysis (Step 200), the large oscillations of higher-mode frequencies begin to occur. Figure 4 shows the axial force in exterior first story column B1. This axial force shows an oscillation comparable to the magnitude of the gravity load itself. Similar behavior can be found in other localized DOFs. Note that although there are several thousand total DOFs in the structural model in OpenSees, the equation of motion solved by UI-SimCor is solved on a condensed system of 470 DOFs, so the massless DOFs are not a factor in the cause of these oscillations.

Note that an alpha factor of 0.05 was used, which is the suggested value based on the UI-SimCor manual (Kwon et al., 2007). This should provide moderate amounts of numerical damping. The alpha factor can range from 0 to 1/3, providing various amounts of numerical damping. Also, the Rayleigh damping is defined with a stiffness proportional component, so the model is significantly overdamped in the high-frequency modes (damping was set to 5% in the 1st and 2nd modes in transverse direction). Even with these damping parameters in place the oscillations are unable to dissipate. If only mass-proportional damping was used, the oscillation would become more significant, since mass-proportional damping results in low damping in short-period (higher-frequency) modes. Therefore, although this study suggests the use of Rayleigh damping in conjunction with the alpha-OS method, it is clear that other mitigation techniques are needed to reduce the likelihood of spurious oscillations in the model results. Note that these oscillations occur even in purely analytical systems, and are therefore not related in any way to experimental errors.

Convergence and static solution of analytical modules

One of the most difficult issues when analyzing large structural systems using SC-HS is that all modules(substructures) are treated equally, with the central coordinator sending displacement commands to be solved statically by each substructure. When one of the substructures contains a large number of effective DOFs, this creates a large and potentially demanding constraint problem which is difficult to solve statically. This can lead to convergence issues when the system is modeled with various nonlinearities. As a result, hybrid analyses with a large number of effective DOFs (masses and loads) can exhibit issues both with high-frequency oscillation, as described above, and with convergence due to the difficult solution of a large constraint problem.

Other hybrid simulations have also shown stability issues with explicit time integration in different architectures using OpenFresco as the coordinator (Del Carpio et al., 2015).

Solutions for using existing SC-HS frameworks for large models

Reducing model DOFs through consolidation and symmetry

To address the problems associated with the use of large models in SC-HS, several mitigation techniques are used in parallel. Since the primary issues stem from the size of the model, the most basic and effective technique for alleviating these issues is by simplifying the analytical model to the extent possible. Note that the techniques described below are primarily meant to resolve issues in SC-HS architectures, such as UI-SimCor, and are less applicable to other methods such as the use of OpenFresco. This is because the primary issues arise in the static solution of the large substructure, not the dynamic alpha-OS solution. The unconditional stability of the alpha-OS method (provided that the stiffness matrix in the formulation is larger than the actual stiffness) dictates that few issues arise in the dynamic solution. Rather, the constraint problem solved statically in a large numerical substructure can easily cause issues. Therefore in SC-HS, it is more important to consolidate masses and gravity loads to reduce the number of *effective DOFs* solved in the equation of motion rather than to remove nodes and elements from the analytical model representing one of the substructures. This addresses multiple issues:

- It cuts down on the amount of communication passed back and forth between the substructures and the coordinator;
- It reduces the number of constraints being applied to the building module, reducing the likelihood of convergence issues;

- With a more coarse distribution of mass, the number of vibration modes (and hence the highest frequencies in the system) becomes lower, somewhat reducing the likelihood or impact of spurious oscillations during the time integration.

However, depending on the purpose of the HS and the reliance on accurate or detailed analytical modeling, there is a limit to how simplified of a model is acceptable. Additionally, although it is often acceptable to lump masses more coarsely in a lateral seismic analysis of a building structure, if such an analysis also aims to achieve reasonable gravity forces, the vertical loads of the system should still be assigned with a more detailed distribution (for correct beam moments, etc.). Since vertical loads are also treated as effective DOFs, they represent a portion of the model complexity which is difficult to reduce or simplify without severely sacrificing the validity of the gravity loading.

Since reduction of model complexity is shown to be one of the most effective ways of achieving better and more stable results, it is also advisable to take advantage of symmetry of the structure and its loadings whenever possible. For instance, since the representative structure is symmetric about line 4 in Figure 1, and the HS consists of ground motion only in the N-S direction, the model can effectively be cut in half, significantly reducing the number of DOFs.

New gravity application for reduction of effective DOFs

In the HS described previously, any DOFs that had gravity loads needed to be included in the effective DOFs. A method is presented here to apply the gravity loading to DOFs that need only exist outside of the coordination software (do not need to be included in the dynamic analysis involving effective DOFs), allowing for the removal of any effective DOFs purely associated with gravity loads. This means that the static loads are moved from P_e to P_s , and in (1) to

(4) $P_s \neq 0$. Following this modification, the effective DOFs include only the DOFs that have mass or are at the boundaries of the substructures.

The gravity analysis is conducted in two steps. In the first step, the loads are applied within the analytical substructure prior to UI-SimCor being initialized with the modules. In this step, all the effective DOFs are constrained from motion (i.e. $u_e = 0$). If the structure were linear, the solution could be found using (3): $u_s = (k_{ss})^{-1}P_s$ (note that k_{ss} is the stiffness matrix of the structure under the condition that $u_e = 0$). In the nonlinear structure studied here, this analysis is conducted iteratively within OpenSees.

Then the modules are initialized within UI-SimCor and the second step of the gravity analysis is performed using a static analysis in SimCor. Obviously, the gravity load vector defined in UI-SimCor is set to *zero*. UI-SimCor will then resolve the unbalanced forces at the effective DOFs, balancing the load incrementally between the substructures. This method proved to work well, with similar success to the original gravity load method implemented in UI-SimCor, but with reduced effective DOFs.

The method described above can have varying levels of efficiency in regards to the reduction of effective DOFs, largely related to the ratio of discrete gravity loading locations to DOFs with mass. For instance in an extreme case, in a building model such as the one used in this study, a typical floor can be composed of 275 nodes. If gravity is distributed throughout all of those nodes based on their tributary areas, but the dynamic analysis is conducted only laterally with rigid diaphragm action, then the effective DOFs in UI-SimCor can be reduced from 276 per floor (275 vertical gravity DOFs and 1 lateral mass at the center of the structure) down to 1 (just the lateral mass), removing 99.6% of the DOFs from the equation of motion. In the case of our structural model and distribution of masses in this system, vertical and lateral masses were

lumped at the column nodes (12 nodes, 24 masses per floor), so this resulted in a 91.6% reduction in the # of DOFs in the equation of motion for this study.

Reducing oscillating through damping and time step size reduction

The above methods are all used in this project to reduce the number of masses and effective DOFs located within the coordination software for SC-HS. Additionally, different forms of damping can be used in an attempt to further decrease the likelihood of spurious oscillations of higher mode behaviors. As discussed earlier, increasing the alpha factor in the alpha-OS method can introduce higher amounts of numerical damping which has been shown to mitigate oscillations in analysis results (Combescure and Pegon, 1997). Through a series of analyses on systems that were showing oscillation, an alpha factor of 0.1 was found to be most effective at reducing that oscillation in this study. Earlier it was discussed that problems (oscillations) can arise associated with mass proportional damping with UI-SimCor analyses due to the low dissipation of high-mode behavior. The stiffness proportional term of the Rayleigh damping causes asymptotically increasing damping ratios as the period of vibration approaches zero. This can help dissipate vibrations that occur in localized modes of the system, which tend to be the problematic areas.

Another solution to the oscillations that can occur in hybrid analyses, is to reduce the time step of the solution. A combination of DOF reduction, optimal structural damping, new gravity load application, and time step reduction, was used to analyze our RC building using SC-HS within the UI-SimCor framework, ultimately ending up with 470 effective DOFs, and a time step of 0.002. As is typical in RC structures, 5% damping was used to form the Rayleigh damping coefficients. For the large-scale HS, this damping ratio was ultimately used in the first lateral and vertical modes of vibration, similar to the damping used by Kim et al. (2011). The

vertical mode of vibration was kept at 5% damping due to the fact that the final HS could result in column failures and subsequent downward vertical acceleration of a line of column nodes. To maintain reasonable results after such a failure, it was important not to have excessively high damping in such a case.

Summary

Ultimately, although moderate success was achieved in running a SC-HS of the representative building structure, a different approach was eventually taken to conduct a HS without many of the issues associated with SC-HS, and will be described in the following section. Although these solution strategies were not ultimately used for this HS, they are presented here due to their ease of use and potential usefulness for other HS with complex analytical models. The solution presented in the following section is much more labor-intensive and may not be appropriate or feasible for other projects. The strategies presented in this section are applicable directly within the current implementation of UI-SimCor, and do not require the reprogramming of communication that will be discussed in the next section. However, given a model of sufficient complexity (such as that used in this HS), the combination of existing techniques (numerical damping, time step reduction) and newly explored strategies (gravity application procedure) cannot completely resolve the issues related to SC-HS.

Dynamic Hybrid Simulation (D-HS) within UI-SimCor

Interface element between OpenSees and UI-SimCor

The methods above inherently either force sacrifices in terms of model complexity and accuracy, or they have limited effectiveness and cannot completely resolve the described issues.

A systematic approach to resolving the issues described above is to eliminate the constraint problem from the HS altogether. Instead of performing a SC-HS, with a central coordinator performing the analysis, the dynamic time integration can be performed directly within the analytical substructure (OpenSees in this case), with the other "modules" treated simply as experimental elements in the OpenSees structural system. Similar to OpenFresco, this type of framework eliminates many of the issues that arise in SC-HS by solving the largest substructure dynamically as opposed to statically, and will from now on be referred to as Dynamic Hybrid Simulation (D-HS). Since the other substructures behave as experimental elements in OpenSees, the tasks performed by the coordination software is reduced to communication: sending displacements to physical specimens and reporting back their restoring force to OpenSees. The benefit of D-HS over SC-HS is that there is no longer a large static constraint problem to be solved. Note that in special cases where multiple substructures are analytical and cannot be included in one model, one of them would still have to solve a set of static displacement constraints to assemble the restoring force vector. The flow of communication in a D-HS is similar to the procedures used in a HS using OpenFresco as a coordinator for OpenSees. This project uses a D-HS architecture that works with UI-SimCor such that the other functions of the UI-SimCor framework could be taken advantage of, such as its user interface, DAQ systems, camera control, etc. Within this architecture, the dynamic time integration is performed in OpenSees as opposed to the coordinator. Specific elements are defined in the model that each communicate with one of the column substructures. These elements, which have been developed by Dr. Oh-Sung Kwon's research group at the University of Toronto, act as interfaces to UI-SimCor, and in OpenSees have an input stiffness matrix which remains constant throughout the analysis. A diagram of the new D-HS communication procedure is shown in Figure 5.

Parallel Processing using OpenSeesMP

When using the standard OpenSees executable (v2.4.4), elements are dealt with in series, meaning that OpenSees will send an element its end displacements and wait to receive restoring forces before moving on to the next element in the model. This is problematic for a HS with multiple substructured elements, because with this limitation of series communication, each experimental column would have to wait for the previous physical specimen to finish executing before continuing. Since the physical application of displacements and measurement of loads typically constitutes the largest portion of time during an analysis step, this limitation can effectively triple the length of a HS if there are three experimental elements, as in our study. Additionally, the existing programming of UI-SimCor dictates that it communicates with all modules in parallel. This contradiction between OpenSees' operation and SimCor's operation causes the programs to hang up when an analysis begins. One systematic way of improving communication between substructures would be to implement a continuous communication similar to those implemented in OpenFresco by Shellenberg et al. (2008).

An effective way to resolve this issue was to use OpenSeesMP, which allows for multi-processor applications, and therefore parallel processing. For this project, each interface element between OpenSees and UI-SimCor and its corresponding nodes are explicitly defined to exist on separate processors, with the remainder of the building structure located on its own processor as well. This allows each experimental substructure to be dealt with in parallel, and for all displacements to be sent to SimCor concurrently, allowing communication to and from the coordinator as normal. Working with OpenSeesMP requires various other considerations regarding recorders, solution algorithms, and other specialized commands, but those are not discussed in this study (see McKenna and Fenves, 2007).

Results of GM Analysis of Full-Scale RC Frame using D-HS

Using the new solution procedure described above, in which the dynamic time integration is performed within OpenSees, a full-scale HS is conducted. The structural system consists of the representative building described earlier, broken into 4 separate subsystems: two first story columns (B1 and A1 in the building plan shown in Figure 1), one 2nd story column (column line B1 in Figure 1), and the remainder of the structure. The three single-column subsystems are represented by physical specimens at the MUST-SIM NEES facility at UIUC, while the 4th subsystem, the remainder of the building, is modeled within OpenSees as described previously. The structural model in OpenSees is modeled as half of the structure, taking advantage of the symmetry of the building and the excitation loading (as described earlier).

The three physical column specimens, located at the MUST-SIM facility at UIUC, were post-tensioned to the strongfloor at their base and to a Load and Boundary Condition Box (LBCB) at their top. The LBCBs in the MUST-SIM lab each consist of a loading plate and 6 actuators, oriented such that they are capable of imposing displacements in all 6 DOFs to the specimens. A diagram of the specimen is shown in Figure 6a, and one of the full specimens at the MUST-SIM lab is shown in Figure 6b, connected to an LBCB.

Model Adjustment based on Column Stiffness

Perturbations of each of the physical columns in all 6 DOFs were used to develop stiffness matrices of those elements, which are used in the OpenSees model to represent the physical columns in the stiffness of the system. Additionally, the information from these stiffness matrices is used to adjust the parameters in the analytical model such that there is consistency across physical and analytical portions of the structural frame. Since the analytical models used

in OpenSees may not have the same vertical and lateral stiffness of the columns tested in the lab, adjustments can be made to the model such that the load distribution among the analytical and experimental columns remain as realistic as possible. For instance, if the test columns were more flexible in the lateral direction than the remainder of the analytical columns on the same story, the physical column would only take a small percentage of the story shear at a given story drift. For this project, the constitutive relationship for the concrete model used in the fiber cross-sections is adjusted (specifically the strain at peak stress) such that lateral stiffness of experimental columns and analytical columns are consistent. As will be shown in the following section, lateral stiffness matching is more important than vertical stiffness matching because relatively large discrepancies in vertical stiffness result in small changes in gravity load distribution, such that matching vertical stiffness is not imperative in this study.

Gravity load application results

The gravity loads in the HS are applied such that the loads are increased incrementally, with a static hold period after the total load is applied, allowing the structure to slowly resolve the unbalanced force and reach equilibrium (while maintaining compatibility). To show the effectiveness of this procedure, the axial forces on columns B1 and B2 (one physical column and one analytical column) are shown from the hybrid analysis in Figure 7, with dashed horizontal lines representing the gravity loads that those same columns hold in a purely analytical solution from OpenSees. The loads developed in the columns are shown to reach their expected values. Note that these loads are not imposed directly to these members, but rather incrementally solved using structural analysis (through applied *displacements*), with the internal gravity forces developing as a result of those displacements (a system-level load application using OpenSees).

Figure 8 shows the vertical displacement at the top of one of those columns (experimental column B1 on the first story). Comparing purely analytical results with the HS shows that the experimental column needed to deform downwards approximately an additional 0.28mm(0.011"), or in other words an additional 25% displacement, to reach equilibrium with the remainder of the structure. However, as shown in Figure 7, this large relative difference in displacement only corresponds to a negligible change in the supported axial force in that column. This is due to the fact that the flexural stiffness of the beams in the frame is significantly smaller than the axial stiffness of the columns. In other words, although the relative change in vertical displacement (25%) between physical and analytical results is large, the absolute change (0.28mm, 0.011") is still far too small for a meaningful amount of shear to develop in the surrounding beams, which would be required for the gravity loads to redistribute elsewhere. Again, this can be concluded from the observation of axial force in B1 due to the system-level solution of the gravity loading. In order for beams to develop additional shear or for surrounding columns to take on additional axial forces, the axial force in B1 would need to drop with respect to the analytical results (which it does not). The conclusion from this observation is that large changes in column vertical stiffness can have a negligible effect on the gravity load distribution in the system.

Earthquake GManalysis results

After imposing gravity loads to the system, and to verify the HS procedure, the structure is subjected to an 85% scaled GM of the east-west recording of the Lucerne station of the 1992 Landers, CA earthquake. The motion is imposed in the transverse direction of the structure. Due to the magnitude of the motion being small early in the GM recording, analyses and simulations are performed beginning 4 seconds into the GM record. The results are compared to the results of a non-hybrid analysis run directly within OpenSees (consistent with the model shown in

Figure 2). A comparison of the time-history of the top building displacement in the two cases is shown in Figure 9. Note that the analysis was started at 4s into the GM due to there being minimal excitation prior to that point. The test is shown up through 13s, after which motions are smaller and dissipate, as the main pulse occurs from 10.5s to 12.5s of the motion. Subsequent figures also present the results during this same time period.

Although this HS does not lead to column failure or structural collapse, the demand on the columns and the other building components is significant. Physical specimens experienced significant flexural cracking, and the beams of the analytical portion of the model developed plastic hinges in many locations. It is therefore important to confirm that the nonlinear behavior of the HS is in agreement with the previous analytical model results to verify your modeling and analysis procedure.

Since it is apparent that global responses should not be affected by the inclusion of physical test specimens into the analysis (since no failures occur), more localized responses are perhaps a better measure of the performance and accuracy of the HS. Figure 10 shows the 1st story shear in the first two frames of the structure (Frames 1 and 2 in Figure 1). Comparing the results of the hybrid test with the analytical results from OpenSees, there is less than a 5% difference between the two cases at peak displacement. This illustrates the legitimacy of the hybrid testing approach, as the shear in frame 1 during the HS is a direct result of shear contributions from two of the test columns and one analytical column in the building model. Figure 11 shows the shear force in column B1 on the 2nd story, one of the experimental test columns. The overall comparison of the two analyses shows consistent results, with a difference at peak of less than 5%.

During the main pulse of the ground motion, Column A1 on the 1st story is subjected to additional compression due to overturning moment. This additional compression, along with the

orientation of the actuators in the LBCB and issues with hydraulics (actuator capacity was lower than expected), caused one of the vertical actuators to hit its force capacity limit prior to the end of the test. Once this limit was reached, LBCB2 was unable to accurately impose the commanded displacements from the OpenSees model onto the test specimen and return the forces to UI-SimCor. Typically problems such as this can abruptly end a HS, as the physical elements in the lab are unable to move forward. However, during this HS a solution was found that allowed the test to proceed using the remaining test specimens.

I-modification and its use for mitigating actuator capacity limits

When introducing physical components to a model and the addition of the associated experimental error, stability issues can arise in HS. Due to limitations in actuator displacement accuracy, deformations within the test structure, and resolution of internal measurement sensors, a physical component within a hybrid analysis may not be subjected to its exact prescribed displacement. This error in displacement obviously results in a likewise error in force reported back to the analytical model. More detailed discussion of this error and its effects can be found in Combescure and Pegon (1997). To combat this limitation of physical control, "I-modification" is used. I-modification is a technique used to correct resulting forces from a physical specimen by multiplying the displacement error (calculated by using higher-precision external measurement sensors) by the stiffness matrix of the physical element, and subtracting that theoretical force error from the measured force value. Typically the initial (maximum) stiffness of the physical element is used, which has the tendency to artificially damp out force errors (Combescure and Pegon, 1997). Note that if there is a large change in element stiffness and/or strength during a test, one must be careful to keep experimental movement errors small, or the I-modification will erroneously introduce an excessively large correction to the resulting forces. Therefore, after

element yielding, damage, or total failure, error tolerances must be tightening in the laboratory software.

Typically I-modification is used as a stabilization term, correcting the force response for small, unavoidable, but measurable errors in physical specimen displacements caused by the limitations of actuator accuracy and precision. However, even when displacement errors are large, provided that the system remains relatively linear, these corrections from I-modification can still be effective in mitigating the potential discrepancies in force response. In this experiment, since there was effectively a loss of control of LBCB2 when one of the actuators hit a capacity limit, the I-modification caused any incremental displacements that were prescribed after the loss of control of LBCB2 to effectively be treated as displacements applied to a linear element, and added this term to the actual force response of the column (under the incorrect displacement). The vertical response of Column A1 on the first story, with and without the I-modification corrections, are shown in Figure 12, and are compared with the computational desktop analysis results. It can be seen that the I-modification effectively mitigates the actuator strength limit, adjusting the restoring force sent back to the model to a reasonable value. This procedure uses every piece of experimental data available after losing actuator control, by only analytically formulating the portion of the force resulting from the displacement error (using I-modification).

Although this actuator capacity limit and subsequent corrections to the restoring force sent back to the analytical model dictate that detailed discussion and analysis of that column beyond this point of the test would be misleading, the resulting response generated from the I-modification corrections show good agreement with the nonlinear OpenSees analytical results. This agreement indicates performing these corrections allows for successful continuation of

testing on the remaining components of the HS. This allows the HS to continue and observations on the remaining two physical specimens to be made.

Overall agreement of HS results with OpenSees data

As shown in Figures 7 through 12, the D-HS using UI-SimCor with OpenSees shows good agreement with a purely analytical OpenSees model, both at the global level, such as top building displacement, as well as at a more local level, such as the column axial and shear forces. Figure 13 also shows the 1st story drift, which perhaps is most affected by the experimental specimens, as they account for a large percentage of the story shear on the 1st story. Again, there is good agreement between the D-HS results and the purely analytical OpenSees results.

Conclusions

A large-scale hybrid simulation is conducted at the MUST-SIM NEES facility at the University of Illinois at Urbana Champaign (UIUC), using the modified UI-SimCor simulation tool and architecture. The complexity of the analytical model required by the objective of this HS was beyond that which has been done in the past using UI-SimCor, which is shown to introduce several difficulties. Multiple mitigation methods are proposed in order to conduct a full-scale hybrid simulation of a 10-story RC structure. Note that the results of a second HS in which experimental columns fail is beyond the scope of this paper and are not discussed here.

When performing a statically-condensed hybrid simulation (SC-HS), where there is a need for static analysis of an analytical substructure, a large and potentially demanding constraint problem will be created if the number of effectiveDOFs becomes large within the analytical substructure. When the analytical substructure is modeled nonlinearly, this constraint problem can lead to convergence issues in implicit integration of the analytical model. Additionally, it is

shown that the alpha-OS method causes large oscillations in the higher-frequency modes of the HS model used in this project. It is shown that for this study, significant reduction of the effective DOFs along with reduction of the time step size, optimal use of numerical alpha damping and high amounts of viscous damping in higher modes are all needed to reduce the convergence and oscillation issues related to the use of a complex analytical substructure in a SC-HS.

A new method of gravity load application for implementation in SC-HS is proposed, in which gravity loads are pre-applied directly within a substructure(s). This procedure effectively condenses out a large portion of the degrees of freedom analyzed dynamically by the coordination software.

Due to the issues caused in the SC-HS of the 10-story RC building model, and the associated need for reduction of model complexity to obtain reasonable results, a different approach is taken to conduct a D-HS, in which the dynamic time-integration is performed within OpenSees. This approach allows for increased model complexity without having issues with large oscillations or having a large potential for convergence issues due to static solution of a large constraint problem.

The multiprocessor capabilities of OpenSeesMP are shown to be effective in significantly reducing the duration of a hybrid simulation in which multiple experimental substructures are needed. Lastly, although loss of experimental control of a specimen (due to actuator limitations) introduces varying levels of error and uncertainty, it is shown that the I-modification technique described by Combescure (1997) can be used to continue structural testing well beyond the limits (force or displacement) of the actuators of a particular specimen, enabling a test to continue with the remaining physical specimens.

Using the assortment of changes and adjustments to UI-SimCor and the hybrid simulation procedure at the MUST-SIM facility, a large-scale HS is conducted using a complex analytical model. The results of the analysis are consistent with the results of a desktop simulation. Since the 85% scaled GM did not produce any failures, the HS are expected to show agreement with analytical results. This test represents a complete system-level response analysis, as the model consists of a 3D RC frame modeling of a 10-story building. The model which was ultimately used in the hybrid simulation has a complexity more than an order of magnitude larger than the previous hybrid tests using similar architecture, representing a step toward more flexibility in the use of hybrid simulations in the future.

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