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NUMERICAL MODELING OF A NOVEL STEEL-CONCRETE COMPOSITE BEAM-COLUMN JOINT SOLUTION

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Abstract. *While Reinforced-Concrete Moment-Resisting Frames (RC MRF) designed according to modern seismic criteria tend to be effective and reliable structural systems, they can be excessively flexible and may reach design drift limits with minimal flexural yielding. This implies that ductility and energy dissipation can be lower than anticipated, leading to higher force and acceleration demands. The "New Performance System" (NPS®) is an innovative solution for improved reliability, efficiency and performance of RC MRFs. The NPS® frame consists of a self-supporting system made of hollow steel columns (with internal pre-installed steel reinforcing bars) and steel truss beams with a steel plate bottom cord. The system is completed with concrete cast on site, taking advantage of the HSS and beam truss elements which double as formwork. Moment continuity at the beam-column connection is achieved by placing deformed, straight steel bars across the beam-column joint through pre-made openings in the HSS columns. Recent experimental campaigns support the notion that properly designed NPS® beam-column joint subassemblies possess desirable seismic performance attributes. The systems are capacity protected and their behavior is controlled by the flexural strength at the beam-column interface, where a flexural hinge forms. Activation of the flexural hinge requires adequate response of the joint panel zone. Herein, a numerical modeling program is undertaken with the VecTor2 software to understand the effect of the prefabricated HSS elements on joint response, as well as to determine an appropriate numerical modeling approach. It is shown that the VecTor2 "steel skin plate" finite element aids accurate strength and deformation predictions of wrapped NPS® joints, and that special attention should be given to the choice of concrete confinement, crack slip, and concrete dilation models.*

Keywords: Beam-Column Joint, Finite Element Modeling, MCFT, Steel-Concrete Composite, Concrete Dilation.

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1 INTRODUCTION

In design of reinforced concrete moment-resisting frames, the choice of beam-column joint solution is a key parameter that influences the extent and type of flexural plastic hinging in the beams, which results in different displacement capacity and energy dissipation [7]. These aspects affect the overall response of the system, including the behavior factor "q" (or "R") that is to be used in design to reduce the seismic action.

While RC MRFs designed according to modern seismic criteria tend to be effective and reliable structural systems, they also present criticalities of both technical and practical nature. For instance, RC MRFs can be excessively flexible and tend to reach design drift limits (e.g. 2.5% drift ratio) without experiencing much of the intended flexural yielding (particularly in tall buildings). This implies that the ductility and the energy dissipated in RC MRFs can be lower than anticipated, leading to higher force and acceleration demands [3]. RC MRFs are also "labor intensive" and relatively slow to construct. For example, they require the erection of formworks and the fabrication of reinforcing cages with substantial need for on-site assembly. The desire to improve the efficiency and performance of RC MRFs led to the development and implementation of many innovative MRF solutions. Among the many suggestions, a novel steel-concrete composite system referred to as "New Performance System" (NPS®) represents a recent and promising solution.

This paper first discusses the unique composition of the NPS® assembly, followed by a summary of the intended seismic behavior. Then, the available experimental research is outlined. Finally, nonlinear finite element models are built in support of a recent experimental campaign. Accurate model performance will be crucial in aiding designers to assess the complete behavior of the NPS® beam-column subassembly, which contains various peculiarities in the joint region, as discussed in the next section. Ultimately, this report provides modeling guidelines for proper representation of the NPS®-type joints. Though all models here are analyzed in VecTor2 the suggestions are equally valid for other finite element program solutions.

2 INTENDED BEHAVIOR OF THE NPS® BEAM-COLUMN SUBASSEMBLY AND EXISTING EXPERIMENTAL RESEARCH

The NPS® frame is a quick-to-construct system that consists of steel Hollow Structural Section (HSS) columns and steel-plate beams, both of which double as structural elements and formwork. The steel-plate formwork acts as the bottom chord of the beam flexural-action resisting system, which also contains inclined plain reinforcing bars welded to the bottom plate and to top-chord steel reinforcement. The self-supporting steel frame is shown during construction in Fig. 1. After placement of the precast floor system (also pictured in Fig. 1), deformed steel bars are positioned through pre-cut openings in the HSS columns to provide moment continuity across the beam-column joint. Along with pre-installed column longitudinal bars and stirrup reinforcement within the HSS, the continuity bars provide one method of transferring shear through the joint. Upon casting of concrete, the composite MRF is complete.

2.1 Intended Behavior of the NPS® system

The frame system is intended to form flexural hinges at the beam-column joint faces, where the moment resistance is provided solely by the continuity reinforcement (since the beam truss top bars and the steel plate bottom chords terminate just before the beam-column interface). The columns are capacity protected to ensure an appropriate frame-yielding mechanism, which requires that the beam-end region possess adequate rotation capacity to absorb the displacement

Figure 1: View of NPS® frame during construction

demand. Furthermore, joint integrity must be maintained throughout deformation.

2.2 Experimental Response of the NPS® System

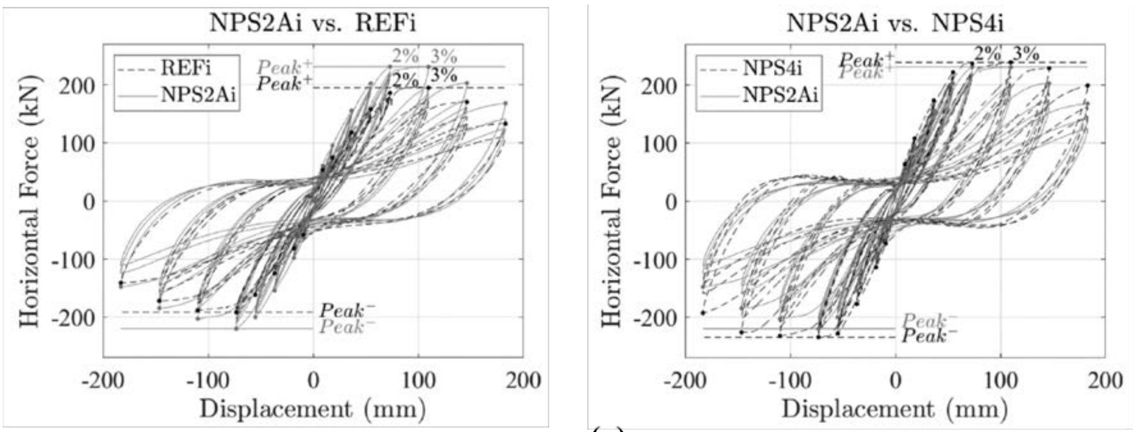
An experimental campaign was carried out by Albright et. al [2] to assess whether the NPS® beam-column joint subassembly actually performs as intended. Specifically, the experimental program sought (i) to identify the behavior of the NPS® system relative to a traditional RC MRF system, and (ii) to investigate the effects of various reinforcing details within the joint. The joint reinforcing details are of particular interest in this paper because the peculiarities of the NPS® system reside within the joint and present numerical modeling challenges. For this reason, only a subset of Albright's [2] experiments specimens are discussed within this work, namely those regarding joint detailing.

The specimens tested by Albright [2] were all designed to be moderately ductile with a q-factor of 3.9, and all have the same nominal dimensions including 3.5 m long columns (with 8 mm thick external HSS column) and 5 m long beams, both with 400 mm square cross-sections. In the design of the test specimens, material strengths were assumed as $f'_c = 45$ MPa for concrete and $f_{sy} = 450$ MPa for reinforcement yielding. The tested subassemblies were grouped into internal and external test groups, though only the internal joint performance is considered herein to respect space limitations. In particular, the results of three tests are summarized in this brief overview. As a control, a traditional MRF reference specimen (REFi) designed according to the Eurocode 8 [5] standards was tested. The other two samples are NPS® joints, one (NPS2Ai) with and one (NPS4i) without a cutout in the out-of-plane direction of the HSS column. The fully-wrapped joint simulates the presence of a 3-D framed joint, but could also be a joint solution of a one-way frame.

Moment continuity was provided through the NPS® joints via 3 top and 3 bottom bars with 30 mm diameter. This choice of large diameter bar was evidently prompted by constructability concerns within the joint, and results in a joint depth-to-bar diameter ratio (h_{col}/d_b) of 13, which is smaller than recommendations provided by for example ACI [1], which are intended to alleviate bond-slip deterioration.

Key global and local response parameters are reported by Albright from a quasi-static reversed-cyclic loading regimen. An overview of the test setup, instrumentation, loading details and calculation of response parameters can be found in the original paper [2].

In terms of global response, the hysteretic force-deformation response was reported for the three specimens, and is copied in Fig. 2 to characterize stiffness loss, energy dissipation and overall ductility.



(a)

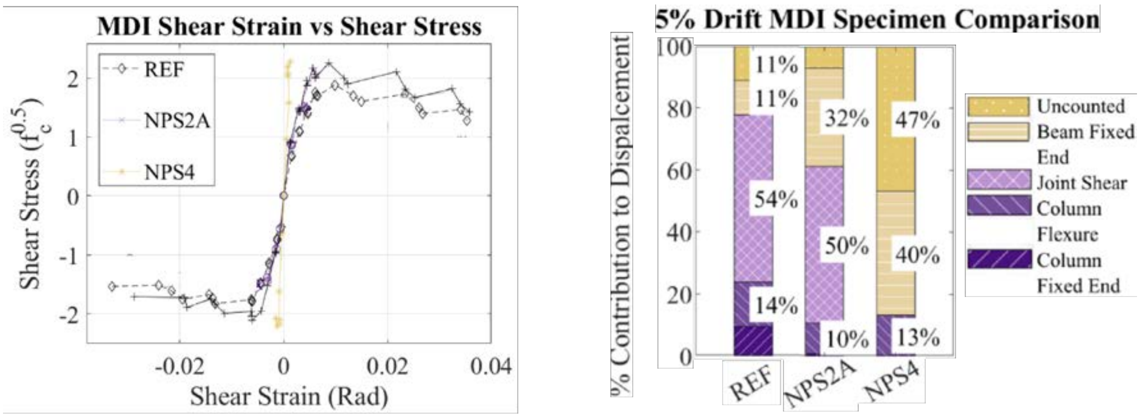
(b)

Figure 2: Experimental (a) hysteresis curves and (b) joint damage, from [2] with permission

Both of the NPS® solutions achieved around a 20% higher strength than that of the REF specimen. Notably, the NPS4i specimen with the wrapped joint maintained a higher residual strength through the 5% drift cycle than either the NPS2Ai or the REFi specimen. Global failure modes were similar for the NPS2Ai and REFi specimens, being characterized by continuity reinforcement yielding around 2% drift followed by severe joint degradation, also shown in Fig. 2. specimen NPS4i, however, experienced no visible joint damage due to the presence of the outer HSS skin, even after removal of the external skin. Instead, since joint failure was inhibited, NPS4i exhibited severe bond-slip degradation which contributed to a highly pinched load-deformation curve.

Locally, the shear deformation and shear stress of the joints was monitored. Many researchers, for example [7], have demonstrated that joint performance is correlated with overall response. The joint shear stress is plotted against the shear strain for each specimen in Fig. 3, indicating a marked reduction in deformability of the wrapped joint. Even at 5% drift, the wrapped joint is perfectly rigid whereas more than 50% of the frame displacement for the traditionally-designed MRF and the unwrapped NPS joint comes from joint shear deformation.

The results of 3b also show that the NPS system trades column-end deformation and joint shear deformation for beam-end hinging, as intended.



(a) **(b)**

Figure 3: Experimental joint responses, from [2] with permission

The results of the experimenal campaign by Albright [2] indicate that the joint confinement provided by the HSS elements can reduce joint deformations, which leads to increased peak and residual strengths as well as improved ductility. However, joint rigidity is accompanied by increased beam-end curvature demand and bond slip of the joint continuity bars. Due to the complex 3D stress states that exist in the joint, an analytical prediction of joint shear deformation and bond stresses is difficult. For example, ASCE41 recommends the use of a semi-rigid joint model, but analysis by Pan et al [12] indicates that such analysis doesn't capture the compression softening effects that occur in damaged joints. Further, properly capturing confinement effectiveness can be difficult for the NPS® system due to the presence of the HSS column. The next sections of this work discuss how nonlinear finite element analysis can be used to analyse the NPS® joints.

3 NUMERICAL MODELING OF THE NPS® BEAM-COLUMN SUBASSEMBLY

The literature on the finite element modeling of beam-column joints is extensive and varied in approach (e.g., [6, 9, 13]), but a consensus exists that shear deformation and bond slip should be included in the model. Among the many available solutions, VecTor2, a general finite element procedure for modeling 2-D structures, is used here, as it has previously produced accurate predictions of RC-MRF performance [13, 20]. VecTor2 uses a smeared rotating crack formulation with a total-load secant stiffness solution algorithm and constituitive models for cracked concrete based on panel experiments conducted at the University of Toronto,. These constituitive models form the basis of the Disturbed Stress Field Model (DSFM) [16]. The program has proven particularly accurate for shear-critical structures.

Among the notable modeling features of VecTor2 relevant to the modeling of beam-column joints (summarized by Wong et al [19]) is the so called ”steel skin plate” element, which allows the user to model sandwich plate reinforcing in the same smeared approach, capturing faceplate buckling and the effects of faceplates on concrete crack widths, crack spacing, concrete tension-stiffening, and out-of-plane confinement effects [17]. This modeling approach is appropriate for the NPS4i specimen, which may be utilized in a scenario in which the joint is not framed by transverse beams. The implementation of the steel skin calls for input of headed stud reinforcement, but if no stud reinforcing is included (as in the case of the NPS® joints) the stud spacing can be set to a very large number. Furthermore, VecTor2 offers bond-slip formulations for both plain and deformed reinforcing, a distinct advantage for modeling of the NPS® system.

VecTor2 also provides various models for refined concrete behavior, notably for analyzing crack slip effects, concrete dilation, and reinforcement bond slip. Bond slip is expected to be an important modeling parameter, given the lower-than-desirable h_{col}/d_b ratio for the NPS® specimens. Generally, the shear stress at which bond slip occurs is a function of confinement. A confinement effectiveness factor of 0.5 is assumed for the REFi and NPS2Ai specimens, while a factor of 1.0 is taken for the well-confined NPS4i joint.

A previous study by Sagbas et. al [13] demonstrated that VecTor2 can accurately reproduce the cyclic response of RC beam-column joints. In that work, it is suggested that perfect bond is unrealistic and bond-slip elements should always be used for embedded reinforcement within the joint. Sagbas [13] also stated that reinforcement could be modeled either discretely or smeared, though a discrete representation leads to better results. Also, smeared out-of-plane reinforcement proved effective in capturing 3-D confinement effects. In terms of constituitive modeling, it is important to capture concrete softening effect (especially in advanced stages of joint deterioration), confinement effects, crack slip, and concrete dilation. The default models of VecTor2 provide these capabilities, though it is shown in the next section that fine-tuning of model selection produces best results for the NPS® joints. Various combinations of relevant material models are listed in Table 1. All other options are set as the default settings.

Behavior	Set 0	Set 1	Set 2	Set 3
Dilation	ν-Limit [11]	ν-Limit	ν-Limit	No ν-Limit [11]
Confinement	Montoya [10]	Kupfer [8]	Montoya	Montoya
Crack Slip	Walraven [18]	Walraven	MCFT [14]	Walraven
Steel Bond	Eligehausen [4]	Eligehausen	Eligehausen	Eligehausen

Table 1: Summary of non-default material models used in VecTor2 joint study

The next section summarizes the performance of the VecTor2 models relative to the experiments outlined in the previous section. A small parameter study is then performed to identify the fitness of the material models in Table 1 for the NPS® joint modeling.

4 VALIDATION OF NUMERICAL MODELING APPROACH

To capture the backbone response of the three specimens, a pushover procedure is performed. The pushover curve provides an envelope of the cyclic response and can help identify global response quantities that are useful in design such as ductility, post-peak strength degradation and failure mechanisms. Initially, material options from set 0 of Table 1 are used, resulting in the purple pushover curves shown in Fig. 4, which are overlayed onto the experimental results. This result captures the initial stiffness and strengths quite well, but over-predicts the peak strength and post-peak behavior for both the REFi and the NPS2Ai specimen. Various other curves are plotted in the figure, indicating material modeling choices that affect the response (corresponding to Table 1). For clarity, if a material set does not produce a noticeable change in response, it is omitted from the figure.

Since post-peak compression behavior (coming from strut action in the joint) is controlled by confinement, an alternative confinement formulation is considered, namely that of Kupfer [8]. Keeping all else the same, this model produces the green pushover curve in Figure 4, which significantly underestimates the observed post-peak behavior for the REFi and NPS2Ai joints. It appears that the joint result is quite sensitive to this parameter.

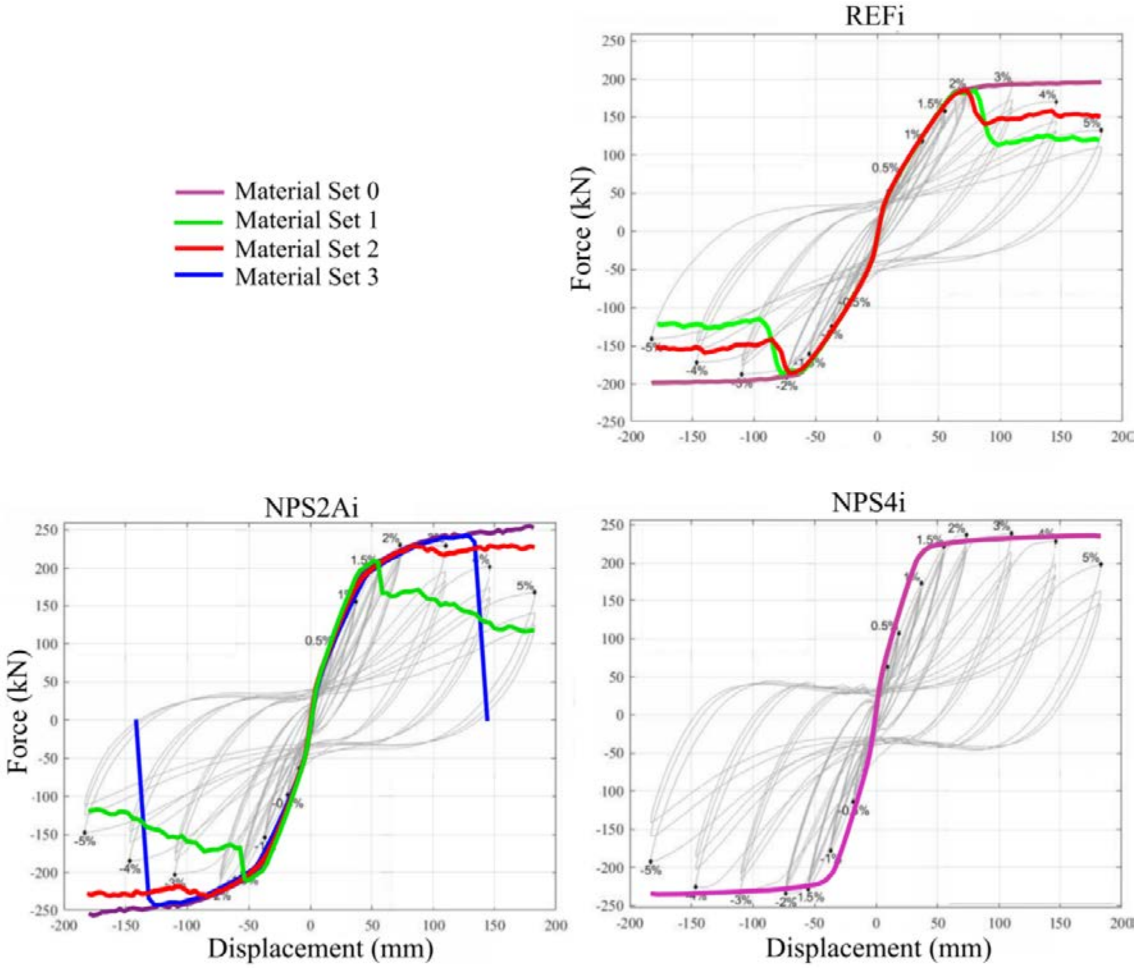


Figure 4: Numerical pushover curves overlayed with experimental joint responses. Material sets that do not affect response are withheld, for clarity.

Next, the effect of changing the crack slip formulation is considered. By default, VecTor2 uses a form of the DSFM in which crack slip is considered. The default model of Walraven [18] calculates the crack slip as a function of the crack width, the concrete strength and the shear stress along the crack face. As discussed by Vecchio [16], allowing slip along the crack face introduces an extra shear strain component and tends to predict a reduced compression softening effect, especially for panels heavily reinforced in both directions, e.g., a beam-column joint. The effect of restricting crack slip for the REFi and the NPS2Ai specimen is shown by the red curve in Fig. 4, where both specimens experience a decrease in post-peak strength compared with the DSFM model. It is noteworthy that restricting crack-slip produces a pushover result that is slightly more aligned with the experimental backbone, though this result should be re-examined for a numerical model that includes cyclic loading.

The crack patterns at 5% drift are shown for each of the specimens in Fig. 5. Given the different damage levels in the joints, various dilation models should be considered. By default, VecTor2 does not limit the poisson ratio ν in the dilation model [15]. This choice appears fine for the REFi joint, which is less damaged than the NPS2Ai joint, and no difference is observed in the response. However, the heavily damaged NPS2Ai joint is predicted rather poorly when no ν-limiter is provided, as shown by the blue curve in Fig. 4. When lateral expansion of concrete is not limited, reinforcement elements that resist the expansion may rupture prematurely, resulting in an artificially brittle failure.

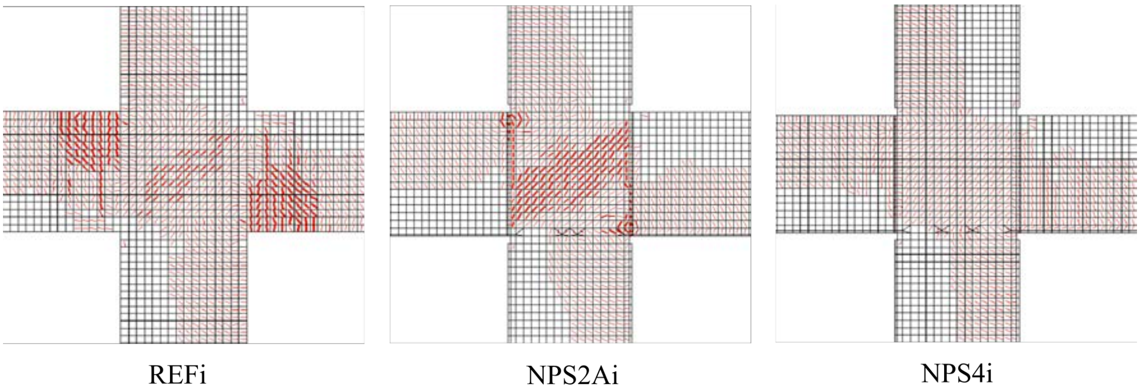


Figure 5: Numerically predicted crack patterns at 5% drift for the three specimens

None of the previous modeling choices have altered the pushover curve of the NPS4i joint, so they have not been plotted in Fig. 4. One peculiar aspect of the NPS4i specimen is that the joint fails due to bar slip, which was also noted by Albright [2]. The bond formulation in VecTor2 [4] relies on an estimation of the confinement pressure factor (defined as $0 \leq \sigma$ / $7.5 \leq 1$). Though it is reasonable to assume the the NPS4i specimen is well-confined with a confinement effectiveness of 1.0, the sensitivity to this assumption is tested by also analyzing the joint for a confinement factor of 0.5. A similar analysis was performed for the NPS2Ai joint, which also exhibited some bond slip, by changing the confinement pressure factor from 0.5 to 0.2. Neither joint is sensitive to the assumption of confinement pressure factor in the bar-slip formulation.

5 SUMMARY AND CONCLUSIONS

In general, the numerical predictions from VecTor2 can capture the initial stiffness and peak response quite well, but the post-peak response is sensitive to a number of modeling assumptions. Even so, the experimental results are consistently replicated quite well up to a drift ratio of 3%, which is larger than design limits set by most codes. Further, the well-confined NPS4i joint is consistently predicted well up to 5% drift, independent of the choices for crack slip, dilation, and confinement.

- Even though the steel corner pieces were not directly modeled in the NPS2Ai specimen, the behavior was still well-captured by the numerical model. This indicates that, for NPS® joints with openings in both directions (and hence small HSS corner pieces), the effects of the HSS on joint performance are minimal. However, this hypothesis can only be verified with a more detailed 3-D analysis that can capture the real geometry of the system.

- The choice of crack slip model appears to affect post-peak joint behavior. Specifically, a no-slip MCFT formulation produces a lower estimate of the post-peak stresses than the

DSFM formulation that includes crack slip in the finite element formulation. This difference between the two formulations is common for heavily-reinforced panels, but it has previously been found [16] that the MCFT formulation underestimates strength, where here the MCFT formulation produces a better estimate of the experimental backbone. The postpeak response has consequences for energy dissipation and should be studied further. This result may be a result of limitations of the pushover procedure and should be re-examined for a model that includes in-cycle degradation.

- Since the experimental specimens displayed significant pinching, it is clear that bond slip of reinforcement is a key parameter that should be included in the model. However, the bond slip models are a function of confinement, which can be difficult to precisely determine. A sensitivity analysis for specimen NPS2Ai, which exhibited the most severely pinched experimental response, shows that strength, stiffness and post-peak response of the pushover curve are not affected by changing the effective confinement. Further effort should be devoted to cyclic modeling in conjunction with a 3-D model to properly capture the true system energy dissipation.

- The steel-skin element, though developed for other applications, shows promise for analysis of joints with external HSS. This would allow a 2-D model to be used for such cases, though some 3-D modeling efforts should be undertaken first to verify that the 2-D simplifications are valid.

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