Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1616

SEISMIC RESPONSE OF OLDER-TYPE REINFORCED CONCRETE CORNER JOINTS

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ABSTRACT

Reinforced concrete buildings constructed prior to the introduction of ductile detailing principles in the 1970s commonly do not have joint transverse reinforcement. Such "unreinforced" joints, especially those at the edges and corners of buildings, have proven vulnerable to damage and collapse in past earthquakes. Test data reveal that the shear strength of such joints is affected by many parameters, including joint area, concrete strength, joint aspect ratio, beam flexural strength, column axial load, and amount and distribution of column longitudinal reinforcement. The ASCE/SEI 41-06 provisions, however, do not reflect the influence of many of these parameters and therefore can result in an inaccurate estimation of shear strength. This paper evaluates the accuracy of those provisions and of alternative formulations incorporating strut-and-tie concepts for estimating unreinforced exterior joint shear strength. In addition, joint axial failure is investigated to understand its relevance to collapse potential in older buildings. The aim of the study is to improve the ability to estimate strength and deformations at onset of both joint shear failure and joint axial failure.

Introduction

Beam-column joints are key components to ensure structural integrity of concrete buildings under seismic loading. Earthquake reconnaissance reveals the apparent vulnerability of these joints if they are inadequately confined by transverse reinforcement. In some cases, failure of older-type corner joints appears to have led to building collapse. Recognizing the importance of beam-column joints, many laboratory tests have been conducted over the last forty years. Most studies, however, were concerned with improving requirements for new joint seismic designs so that they have adequate strength and ductility. Fewer studies focused on seismic performance of older-type exterior joints lacking transverse reinforcement. The aim of this study is to improve the understanding of strength characteristics of older-type corner beam-column joints so that improved building seismic assessments can be done.

The study reported here builds on test data and analytical modeling developments for

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beam-column joints. Test data were gathered from numerous sources identified in the literature. Some analytical strength models of reinforced joints based on the softened strut-and-tie concept have been presented by Parker and Bullman (1997), Vollum and Newman (1999), Hwang and Lee (1999), and Wong (2005). Furthermore, ASCE/SEI 41-06 (referred to hereafter as ASCE 41) specifies a shear strength model for unreinforced joints. Alternative analytical strength models for unreinforced joints based on the strut-and-tie concept also are briefly presented. We also discuss the very limited test data on the axial load failure of beam-column joints.

Failure of Exterior/Corner Joints under Earthquake Loading

Beam-column joints play a key role in the framing action of beam-column frames and also are essential for transmission of column axial loads. Earthquake reconnaissance reports identify cases of building collapse due to the failure of beam-column joints. Two typical examples are illustrated in Fig. 1. Figure 1(a) shows the collapse of the Kaiser Permanente building in Granada Hills. The severely damaged state of the beam-column corner joints gives the appearance that failure of these joints may have triggered the partial collapse of this building. Figure 1(b) shows a building that sustained partial collapse during the 1999 Izmit, Turkey earthquake. Again, the severe damage of the beam-column joints and partial loss of axial load continuity at a lower-level beam-column joint suggests the potential for joint failure to contribute to building collapse.

(a) Kaiser Permanente Building, Northridge earthquake, 1994 (photo courtesy of G. Edstrom)

(b) A Building in Turkey, Izmit earthquake, 1999 (Sezen et al. 2000, Engindeniz 2008)

Figure 1. Building collapses due to beam-column joint failure.

To simulate the failure of unreinforced beam-column joints, many laboratory tests have been done on joints with different geometries, materials, reinforcement ratios and details, and column axial loads. The majority of these tests were on planar exterior joints; few corner joints have been tested. These tests show that joint failure can significantly reduce the seismic performance of moment resisting frames. For this reason, procedures for incorporating the nonlinear behavior of joints in analytical models should be pursued (Mosalam et al., 2009).

Current Joint Shear Strength Assessment Approaches

Most current standards and codes have recommendations for joint shear strength of ductile joints, and some have recommendations for unreinforced joints. Joint shear strength provisions in ASCE 41 are appealing for professional practice because they are familiar and simple to implement. According to ASCE 41, nominal joint shear strength is defined as

$$V_n = \gamma \sqrt{f_c^{'}} b h_c \tag{1}$$

where γ is a coefficient, $f_c^{'}$ is concrete compressive strength in psi, h_c is total column depth, and b is effective joint width defined by either ACI 318-08 or ACI 352-02. The ACI 352-02 definition of effective joint width is adopted in this paper. The coefficient γ is a function of joint geometry (that is, whether it is of exterior, interior, or knee configuration, and whether there is a beam framing into the joint in the orthogonal direction). For a joint with column above and below and a beam framing into one face, $\gamma = 6$.

Past laboratory test data were collected from the literature. Figure 2 presents some results for unreinforced joints having column above and below with a beam framing into one face. For each test an envelope (backbone) curve was developed. Nominal yield displacement was defined by the intersection of a secant through the envelope 75% of the maximum load and a horizontal line at the maximum load. Ultimate displacement was defined as the displacement at which resistance decayed to 85% of the maximum. Nominal displacement ductility μ_D, defined as the ratio of ultimate to yield displacement, ranged from approximately 1.5 to 5 (Fig. 2.a). The measured strength coefficient γ ranged from approximately 5 to 14, compared with $\gamma = 6$ in ASCE 41. There is no clear relation between γ and μ_D.

Joint shear strength decreases with increasing joint aspect ratio (that is, increasing beam to column depth) (Fig. 2.b). Assuming that joint shear is resisting primarily by a diagonal compression strut across the joint, a joint with high joint aspect ratio would require a steeper strut, which may explain the trend.

Three different failure modes are reported for the laboratory tests, namely joint shear failure before beam yielding (designated J), joint shear failure after beam yielding (designated BJ), and joint shear failure after column yielding (designated CJ). (Tests having beam bar straight anchorages without hooks, which may experience excessive bar slip, are excluded.) In the case of beam or column yielding, the apparent joint shear strength is the shear corresponding to yielding of the framing member. Thus, joints with yielding framing members tend to have apparent joint shear strength less than members without yielding framing members (Fig. 2.b).

Considering this observation, one approach to analyzing the data would be to separate the two failure types, then for each type develop a separate model.

An alternative approach is to analyze all the data together, considering beam longitudinal reinforcement as a variable that determines joint shear strength. Figure 3 presents the data from this perspective, showing that joint strength varies with joint aspect ratio and with beam longitudinal reinforcement ratio. Walker (2001) and Alire (2002) previously noted the reinforcement ratio effect for interior beam-column joints. In subsequent discussion, the variable on the horizontal axis in Fig. 3 is referred to as the beam reinforcement index.

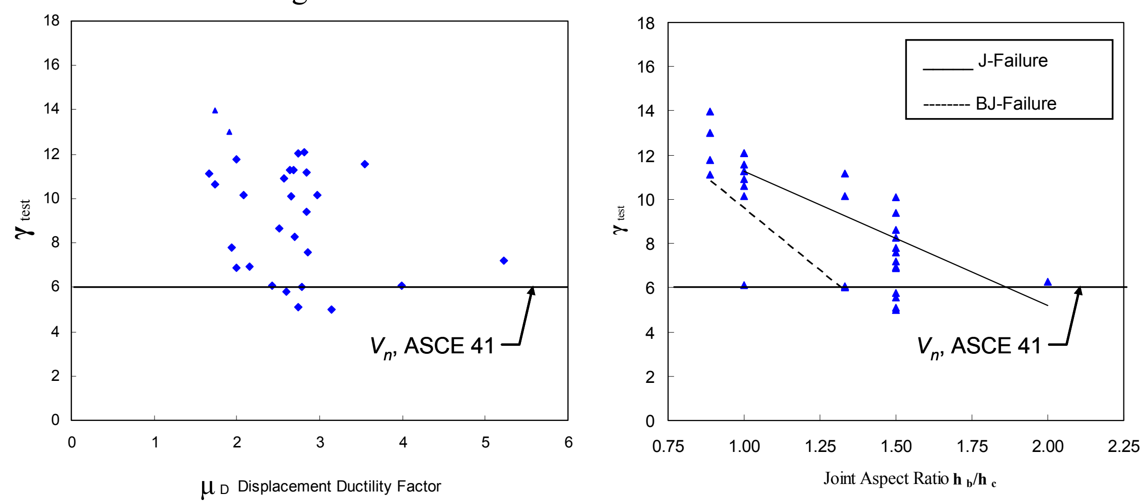

Figure 2. Assessment of ASCE 41 provisions for isolated exterior joint shear strength against experimental results.

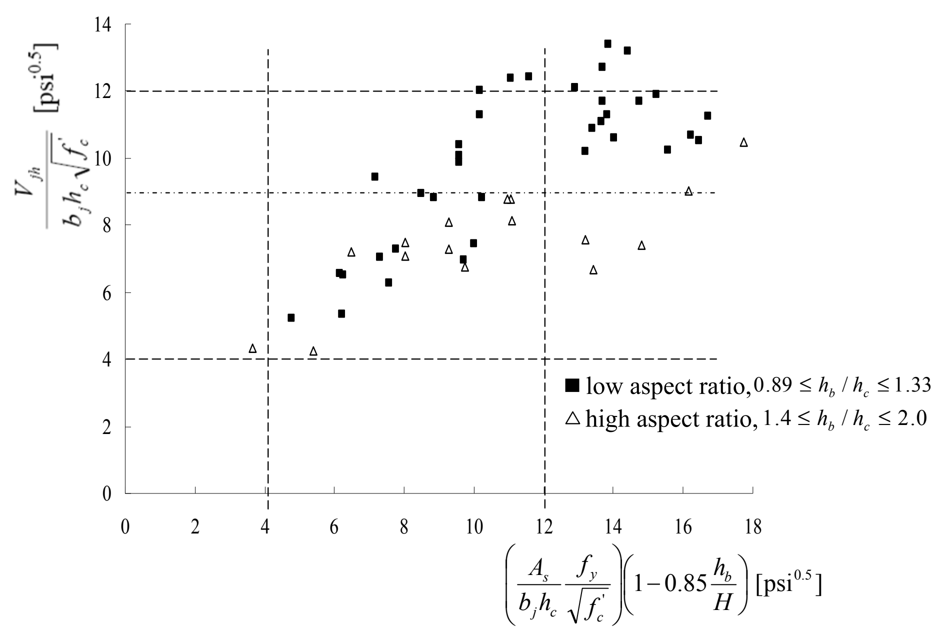

Figure 3. Effect of beam reinforcement.

The preceding observations suggest that alternative formulations of joint shear strength may improve strength predictions relative to the ASCE 41 formulation. Some alternative formulations are presented in subsequent sections of this paper.

Alternative Joint Shear Strength Assessment Approaches

The strut-and-tie concept has been used successfully to estimate shear strength of reinforced beam column joints by Hwang and Lee (1999) among others. The following text introduces two analytical models for unreinforced exterior joints based on the strut-and-tie concept. The study is limited to the case of exterior joints with continuous column and a beam framing into only one joint face. The models consider effects of joint aspect ratio, failure mode, axial load, and yield penetration of beam reinforcement.

Model 1: ACI-318 Softened Strut and Tie Model

If a beam or column yields prior to joint shear failure, longitudinal reinforcement yielding penetrates the joint, leading to concrete dilation and reduced joint shear strength. This is believed to be the reason why apparent joint shear strengths are less for BJ and CJ failure modes than for J failure modes (Fig. 2 and 3). Thus, the joint shear strength for BJ and CJ failure modes is readily determined as the joint shear corresponding to development of framing member strength. The remaining question for such joints is the deformation at which joint failure occurs (this subject is not further pursued here). For joints sustaining J-type failure, that is, joint shear failure before yielding of adjacent framing members, we can investigate use of a strut-and-tie model to predict strength. Here we use the ACI 318-08 strut-and-tie model.

The effective strut compressive strength f_{cu} and diagonal strut capacity D are

$$f_{cu} = 0.85\beta_s f_c^{'} \tag{2}$$

$$D = f_{cu} A_{str} \tag{3}$$

where β_s is concrete softening coefficient defined in ACI 318-08. For the case of a bottle-shaped strut with no crack control reinforcement, $\beta_s = 0.6$. A_{str} is the concrete strut area calculated by

$$A_{str} = a_s b_s \tag{4}$$

where b_s is joint width defined by ACI 352-02 and a_s is the strut depth defined as

$$a_s = \sqrt{a_b^2 + a_c^2} \tag{5}$$

in which a_b and a_c are the compression zone depths of the beam and column. The quantity a_b is defined by the familiar expression for neutral axis depth of a cracked, linear beam as

$$a_b = kd_b \tag{6}$$

$$k = [(\rho+\rho')^2 n^2 + 2(\rho+\rho'\frac{d_b'}{d_b})n]^{1/2} - (\rho+\rho')n \tag{7}$$

where d_b and d'_b are depths from extreme compression fiber to centroids of beam tension and compression longitudinal reinforcement, n is the modular ratio, and ρ and ρ' are beam tension and compression reinforcement ratios. The quantity a_c can be estimated by

$$a_c = (0.25 + 0.85\frac{N}{f_c' A_g})h_c \qquad \le \; 0.4\, h_c \tag{8}$$

where N is column axial load and A_g is gross section area. The limit on a_c not to exceed $0.4h_c$ was calibrated from the test data.

The joint shear strength is

$$V_{jh} = D\cos\theta \tag{9}$$

$$\theta = \tan^{-1}(\frac{d_b - d_b'}{d_c - d_c'}) \tag{10}$$

where d_c and d'_c are depths from extreme compression fiber to centroids of tension and compression longitudinal reinforcement in the column. After obtaining joint shear strength from Eq. 9 corresponding to J-Failure mode, it should be compared with that corresponding to flexural yielding of beam or column, and the least value should be used as a limiting shear strength of the joint.

Figure 4 compares values of the coefficient γ obtained from tests and from Eq. (9).

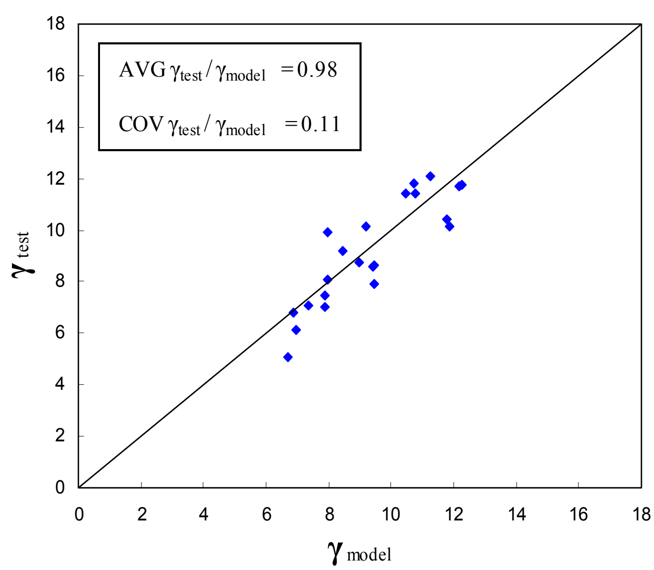

Figure 4. Strut-and-tie model predictions of shear strength of unreinforced isolated exterior joints experiencing J-Failure mode, Model 1

Model 2: Empirical Shear Strength Model

In Model 1, the test results were split into two bins. In one bin were J-failure types, that is, joints that failed before yielding of the framing member. In the second bin were cases where joint failure occurred after yielding of the framing member. Flexural failure determined the strength of the latter bin, whereas the strut-and-tie model determined strength in the former bin. We now consider a new model, referred to as Model 2, in which the two bins are considered together.

In Model 2, we define a beam reinforcement index, which is derived considering the free body diagram in Fig. 6. The horizontal joint shear is expressed as

$$V_{jh} = A_s f_s - V_c = A_s f_s - \frac{L + h_c/2}{H} V_b = A_s f_s \left(1 - \frac{L + h_c/2}{H} \frac{jd_b}{L}\right) \tag{11}$$

where A_s and f_s are the total cross-sectional area and stress of tension beam reinforcement, V_c and V_b are the column and beam shear forces, h_b is the beam depth, H is the height between upper and lower column inflection points, L is the length from the beam inflection point to the column face, and jd_b is the internal moment arm of the beam.

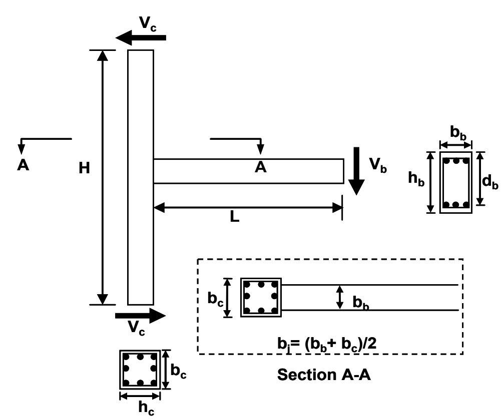

Figure 5. Global free body diagram.

To simplify Eq. 11, the following approximation is made,

$$jd_b = 0.8h_b \Rightarrow \left(\frac{L + h_c/2}{H}\right)\left(\frac{jd_b}{L}\right) = \left(\frac{L + h_c/2}{L}\right)\left(\frac{0.8h_b}{H}\right) \approx 0.85 \frac{h_b}{H} \tag{12}$$

If beam reinforcement is yielding, $f_s = f_y$ can be used in Eq. 11 assuming that the material of the beam reinforcement is elastic-perfectly-plastic. Dividing Eq. 11 by $b_j h_c \sqrt{f_c^{'}}$, the beam reinforcement index is obtained,

$$\frac{V_{jh}}{b_j h_c \sqrt{f_c^{'}}} \approx \left(\frac{A_s f_y}{b_j h_c \sqrt{f_c^{'}}}\right)\left(1 - 0.85 \frac{h_b}{H}\right) \tag{13}$$

Finally, the joint shear strength equation is proposed as

$$\frac{V_{jh}}{b_j h_c \sqrt{f_c^{'}}} = \Phi\left[\left(\frac{A_s f_y}{b_j h_c \sqrt{f_c^{'}}}\right)\left(1-0.85\frac{h_b}{H}\right)\right] \geq \alpha_1 \frac{\cos\theta}{1.31+0.085\left(\dfrac{h_b}{h_c}\right)} \tag{14a}$$

$$\leq \alpha_2 \frac{\cos\theta}{1.31+0.085\left(\dfrac{h_b}{h_c}\right)} \tag{14b}$$

where $\theta = \tan^{-1}(h_b/h_c)$, Φ is the over-strength factor due to strain hardening of the beam reinforcement, α_1 is 10 in [lb and in. units] and 0.83 in [N and mm units], and α_2 is 23 in [lb and in. units] and 1.91 in [N and mm units]. For simplicity, Φ is assumed to be 1.25 at the minimum joint shear strength and decreases linearly to $\Phi = 1.0$ at the maximum joint shear strength.

The procedure to predict the joint shear strength by the proposed model is summarized as follows:

(1) Input the joint geometry, concrete strength, and joint aspect ratio.
(2) Determine the minimum, Y_{min}, and maximum, Y_{max}, joint shear strengths as shown in Fig. 6.
(3) Calculate the beam reinforcement index by Eq. 13.
(4) Check if the calculated beam reinforcement index is located between X_1 and X_2. If so, interpolate for the corresponding over-strength factor, Φ, as shown in Fig. 6.
(5) Calculate the joint shear strength by Eq. 14.

Figure 7 compares values of the coefficient γ obtained from tests and from Eq. (14).

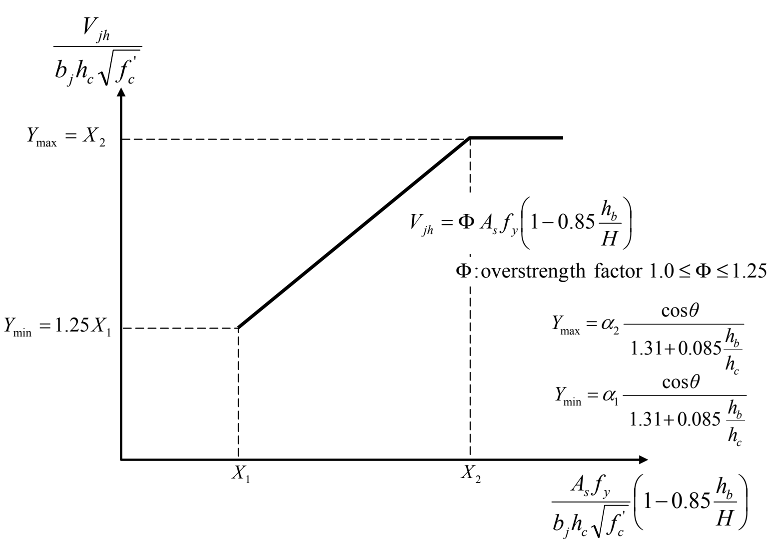

Figure 6. Empirical Model 2.

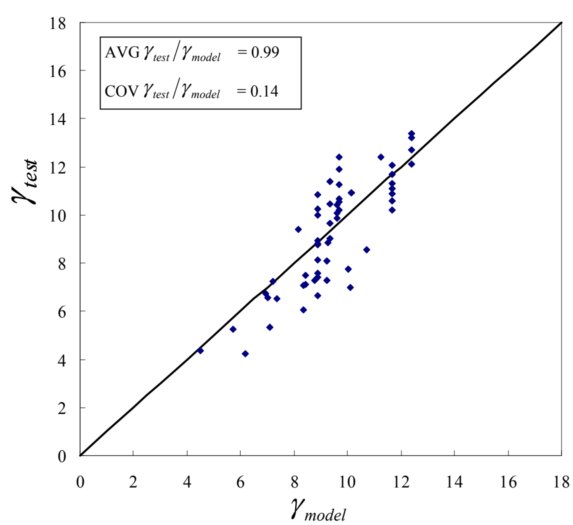

Figure 7. Empirical Model 2 Verification

Axial Collapse Potential of Beam Column Joints

Few tests have continued to the point of axial collapse of the joint, as judged by vertical shortening through the joint or adjacent column regions. Figure 8 contains a sampling of data for unreinforced joints (Moehle, 2003). From these tests it appears that exterior joints may be susceptible to axial collapse under very large drifts or under high axial loads. More data are needed to draw more definitive conclusions about collapse propensity for unreinforced joints.

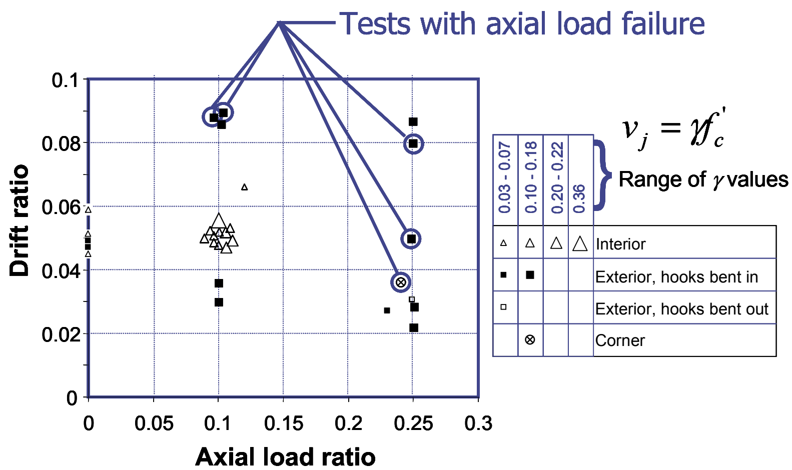

Figure 8. Laboratory data for unreinforced joints with and without axial failure (Moehle, 2003).

Conclusions

Behavior of unreinforced exterior beam-column joints under earthquake loading effects is discussed, with the following observations.

1) Based on photographs of buildings collapsed or heavily damaged in past earthquakes, a plausible conclusion is that failure of beam-column joints, especially at the exterior of a building, can lead to building collapse during earthquake shaking.
2) The ASCE 41 provisions for shear strength produce conservative estimates of strengths observed in laboratory tests on exterior joints.
3) Shear strength of unreinforced exterior joints is strongly affected by joint aspect ratio and flexural strength of the members framing into the joint. Two joint strength models introduced in this paper incorporate these effects and produce joint strength estimates that correlate closely with measured strengths.
4) Axial failure of unreinforced exterior joint is plausible at large drifts or under high axial loads.

Acknowledgments

This study is supported by the NSF award #0618804 through the Pacific Earthquake Engineering Research Center. Opinions, findings, or recommendations expressed in this paper are those of the authors and do not necessarily reflect those of NSF.

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