

# Cold-Formed Steel Special Bolted Moment Frames: Cyclic Testing and Numerical Modeling of Moment Connections

by

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## ABSTRACT

Cyclic tests on nine full-scale beam-column subassemblages were carried out in support of the development of a new lateral load-resisting system recently introduced in AISI-S110: *Standard for Seismic Design of Cold-Formed Steel Structural Systems—Special Bolted Moment Frames*. With double channel beams and HSS columns interconnected by bearing-type high-strength bolts, all specimens showed a story drift capacity significantly larger than 0.04 radian. Typical response is characterized by a linear response, a slip range, followed by a significant hardening region due to bolt bearing. Three failure modes were identified. Confining in the connection region, inelastic action through bolt slippage and bearing is ductile and desirable. Such inelastic action always occurs first, but either column or beam may also experience buckling. Beam buckling is most undesirable due to significant post-buckling strength degradation. Extending the concept of instantaneous center of rotation of an eccentrically loaded bolt group, a model that can reliably simulate the cyclic behavior of the bolted moment connection is presented.

## INTRODUCTION

The American Iron and Steel Institute (AISI) is in the process of developing a seismic design standard for cold-formed steel, *Standard for Seismic Design of Cold-Formed Steel Structural Systems—Special Bolted Moment Frames - AISI S110* [AISI, 2007]. The first seismic force resisting system introduced in the AISI seismic standard is termed Cold-Formed Steel—Special Bolted Moment Frames (CFS—SBMF). It is common that this type of frames is composed of cold-formed

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Hollow Structural Section (HSS) columns and double-channel beams. Beams are connected to the column by using snug-tight high-strength bolts.

The first objective of this study was to identify through cyclic testing both the desirable limit state that can be counted on to dissipate energy in a stable manner and other limit states that should be avoided in design through the capacity design principles. The second objective of this study was to develop a mathematical model of the observed bolted connection cyclic behavior that can be used for predicting maximum forces that can be developed in moment connection for capacity design purposes [Sato and Uang, 2008].

### TEST PROGRAM

Figure 1(a) shows the test setup for the testing of beam-column subassemblies. Each specimen was composed of a column and a half-span beam on each side of the column. For testing purposes, the specimen was rotated 90 degrees. A total of nine full-scale beam-column subassemblies were tested (see Table 1). For each specimen the beam (ASTM 607 Class 1, Gr. 50 steel) was connected to the column (A500 Gr. B steel) by eight 25.4 mm (1 in.) diameter, bearing-type SAE J429 Grade 5 high-strength bolts, which were equivalent in mechanical properties to ASTM A325 bolts, in standard holes [see Figure 1(b)].

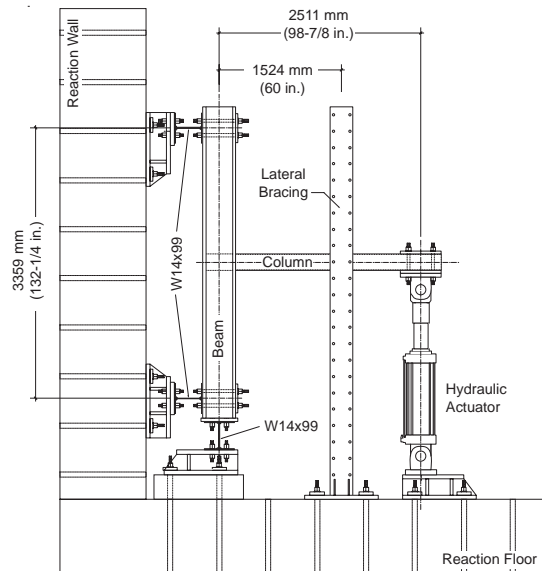


FIGURE 1(a) – TEST SETUP

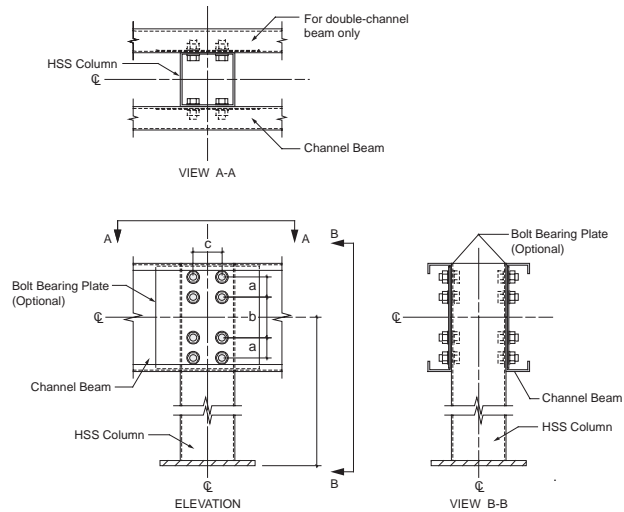


FIGURE 1(b) – BOLTED MOMENT CONNECTION

TABLE 1(a) – MEMBER SIZE

Specimen No.	Beam, mm	Column, mm	Bolt Bearing Plate, mm
1, 2	2C305×89×2.7 (2C12×3½×0.105)*	HSS203×203×6.4 (HSS8×8×¼)	3.4 (0.135)
3	2C406×89×2.7 (2C16×3½×0.105)	HSS203×203×6.4 (HSS8×8×¼)	N/A
4	2C406×89×2.7 (2C16×3½×0.105)	HSS203×203×6.4 (HSS8×8×¼)	3.4 (0.135)
5, 6, 7	2C406×89×3.4 (2C16×3½×0.135)	HSS203×203×6.4 (HSS8×8×¼)	N/A
8, 9	2C508×89×3.4 (2C20×3½×0.135)	HSS254×254×6.4 (HSS10×10×¼)	N/A

\* Dimensions in inch.

TABLE 1(b) – BOLTED CONNECTION CONFIGURATION

Specimen No.	a*, mm	b, mm	c, mm
1, 2	64 (2½)*	76 (3)	108 (4¼)
3, 4, 5, 6, 7	76 (3)	152 (6)	108 (4¼)
8, 9	76 (3)	254 (10)	159 (6¼)

\* Dimensions in inch, \*\* See Figure 1(b).

A combination of displacement transducers, inclinometers, strain gage rosettes, and uniaxial strain gages were used to measure global and local responses [Hong and Uang, 2004]. The loading sequence specified in the AISC Seismic Provisions [AISC, 2005] for steel beam-to-column moment connection test was imposed to the column tip to simulate the story drift.

## **TEST RESULTS**

The global response of all specimens was similar. The cyclic behavior was dominated by the slip-bearing action in the bolted connection in a story drift up to 4%. Beyond this drift level, the specimens eventually failed in either beam buckling, column buckling, or excessive bearing deformation in the bolted connection, depending on the relative strength of these structural components.

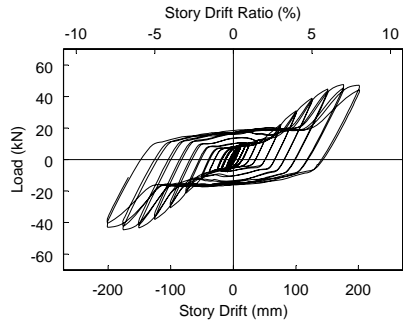
### **Connection Failure**

Specimen 3 did not experience yielding or buckling in the beam and column. Instead, the specimen was able to sustain a stable hysteresis response up to a story drift of 8% [see Figure 2(a)]. Such global response, which can also be identified in all other specimens, is characterized by three regions. Initially, the subassembly responded elastically with the bolted connection acted as a rigid joint. Once the friction resistance of the bolted connection was overcome, a plateau in the measured response due to bolt slippage resulted. The third region showed a significant hardening in strength once the bolts started to bear against the beam and column elements. Figure 2(b) shows components of the story drift due to beam, column, and connection deformations. Note that the contribution from the bolted connection (i.e., slip-bearing) was significant.

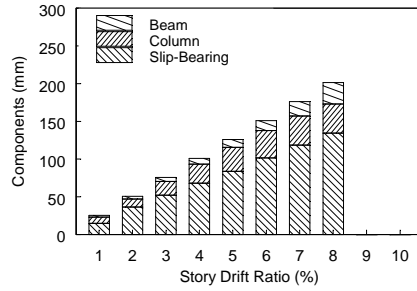
### **Beam Buckling**

Specimens 1, 2, and 4 experienced beam local buckling. Two beam sizes were used to study the effect of the flat depth-to-thickness ratio ( $w/t$ ) of the beam on the cyclic response.

The global response of Specimens 2 and 4 are shown in Figure 3. (The response of Specimen 1 is similar to that of Specimen 2 and is, therefore, not presented.) Beam buckling in Specimen 4 was very severe [see Figure 4(b)], which resulted in a drastic drop in strength. For Specimen 2, web local buckling (WLB) was first observed at 6% story drift. But strength degradation did not occur until flange local buckling also developed at 10% drift [see Figure 4(a)]. Although beam buckling occurred at a very large drift level, it appears prudent to limit the  $w/t$  ratio to 150, which corresponds to  $6.18\sqrt{E/F_y}$ , to control WLB.

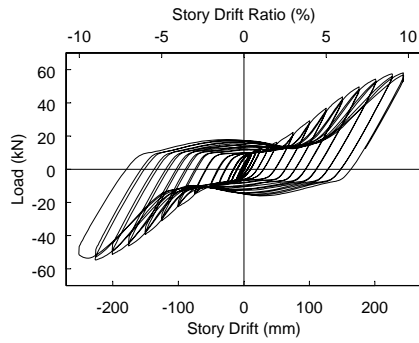


(a) – Hysteresis Response

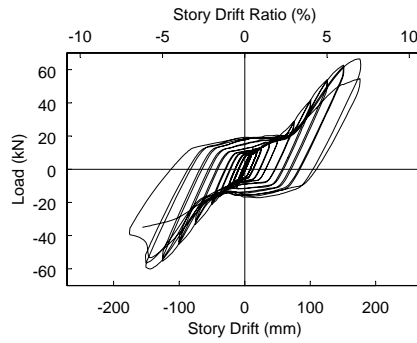


(b) – Story Drift Components

FIGURE 2 – BOLTED MOMENT CONNECTION



(a) – Specimen 2 ( $w/t = 109$ )



(b) – Specimen 4 ( $w/t = 147$ )

FIGURE 3 – GLOBAL RESPONSE OF SPECIMEN 2 AND 4



(a) Specimen 2 at 10% Story Drift



(b) Specimen 4 at 7% Story Drift

FIGURE 4 – BEAM LOCAL BUCKLING

### Column Buckling

A total of five specimens experienced column local buckling. The first group (Specimens 5, 6, and 7) had the same size column as Specimen 3, but a larger beam size was used to force column buckling. The second group (Specimens 8 and 9) had larger beams and columns.

The typical global responses from each group are presented in Figure 5. Figure 6 shows the observed column local buckling mode. Local buckling of Specimen 7 was first observed at 7% story drift. But the specimen was able to respond in a stable manner until 9% drift. Specimen 9 experienced local buckling at 4% story drift. But the higher flat width-to-thickness ratio ( $w/t = 40$ ) of the column caused the strength to degrade drastically beyond 5% story drift. To avoid significant strength degradation, however, it appears prudent to limit the  $w/t$  ratio to 40, which corresponds to  $1.58\sqrt{E/F_y}$ , to control column local buckling.

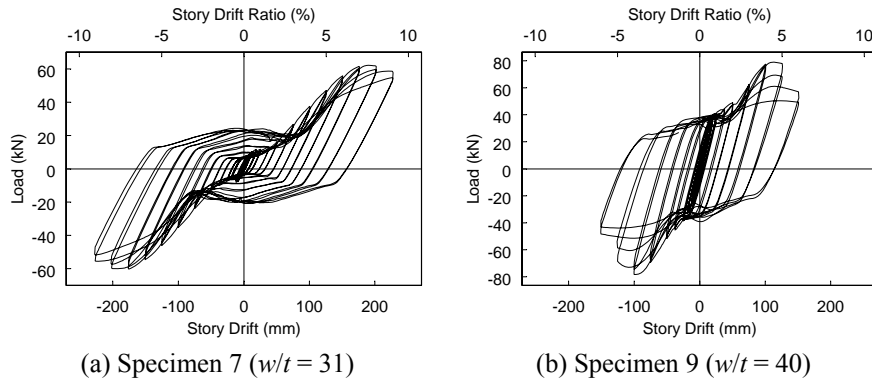


FIGURE 5 – GLOBAL RESPONSE OF SPECIMEN 7 AND 9

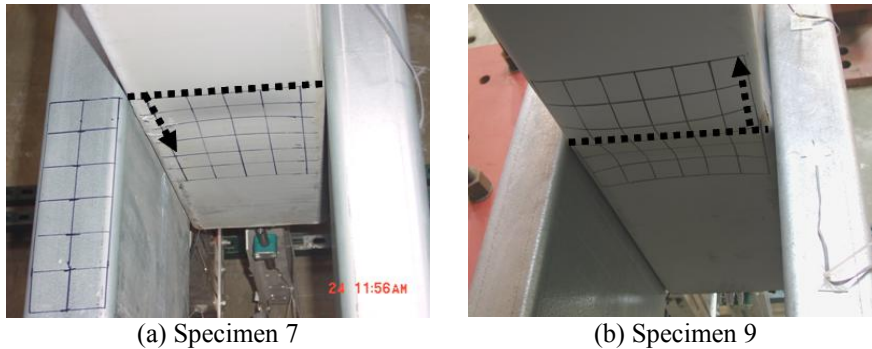


FIGURE 6 – COLUMN LOCAL BUCKLING

## EVALUATION OF SEISMIC FORCE RESISTING MECHANISM

The global response of all specimens in the practical drift range of interest was governed by the inelastic action in the bolted moment connection. Under lateral load, the bolt group in a CFS–SBMF is subjected to an eccentric shear (Figure 7). The bolted connection first responds in the elastic range, which is then followed by slip, hardening, and unloading in each excursion. Slip occurs when the friction resistance ( $R_S$ ) of individual bolts is overcome:

$$R_S = kT \quad (1)$$

where  $k$  = slip coefficient, and  $T$  = snug-tight bolt tension. The slip range depends on the oversize of the bolt holes. Once the bolts are in bearing, hardening would occur. The bearing resistance ( $R_B$ ) of individual bolts can be expressed by the following formula [AISC, 2005b; Fisher, 1965]:

$$R_B = R_{ult} \left[ 1 - e^{-\mu(\delta/25.4)} \right]^\lambda \quad (2)$$

where  $\delta$  = bearing deformation (mm),  $R_{ult}$  = ultimate bearing strength,  $e = 2.718$ , and  $\mu, \lambda$  = regression coefficients. In the bearing range, the resistance of individual bolts includes both friction and bearing resistances (i.e.,  $R = R_S + R_B$ ).

The coefficients and snug-tight bolt tension force assumed in this study are summarized in Table 2. Lacking data to derive coefficients [Fisher et al., 1963; Crawford and Kulak, 1968; Kulak et al., 2001], the tabulated values were shown to provide good correlation with the test results in this study.

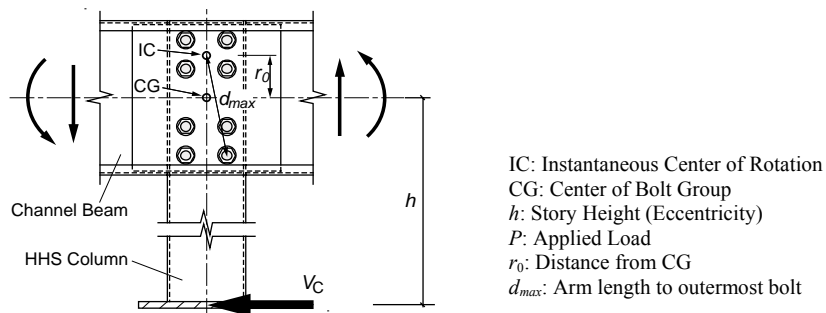


FIGURE 7 – BOLT GROUP IN ECCENTRIC SHEAR

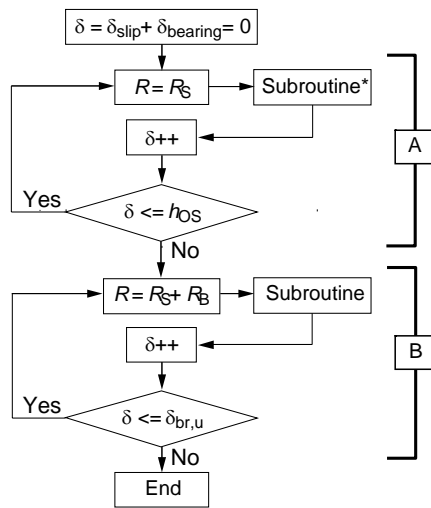
TABLE 2 – ASSUMED COEFFICIENTS AND BOLT TENSION FORCE

Specimen No.	$k$	$T$ , kN	$\mu$	$\lambda$
1 to 7	0.33	44.5 (10) <sup>a</sup>	5	0.55
8, 9		91.0 (21)		

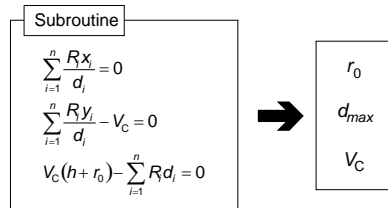
<sup>a</sup> Snug-Tight Bolt Tension in kips.

## MONOTONIC LOADING ANALYSIS

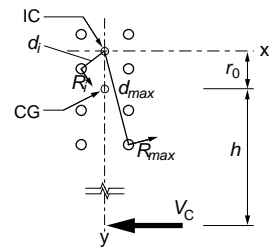
Monotonic analysis can be used to establish the response envelope as observed from cyclic testing. Referring to Figure 7 for the bolt group in eccentric shear, a strength analysis based on the instantaneous center (IC) of rotation theory was used [Crawford and Fisher, 1971; Salmon and Johnson, 1996]. Figure 8 shows the numerical algorithm, where part A deals with the response in the slip range and part B deals with the response in the hardening range.  $h_{os}$  [= 1/16 in. (= 1.6 mm)] in the flowchart refers to the hole oversize. The typical predicted response envelopes for Specimens 2, 3, and 7 are shown in Figure 9. The predicted response envelop shows a very good agreement with the experimental results.



(a) Flow Chart



(b) Subroutine for Force Equilibrium  
 FIGURE 8 – NUMERICAL ALGORITHM



(c) Bolt Group Force



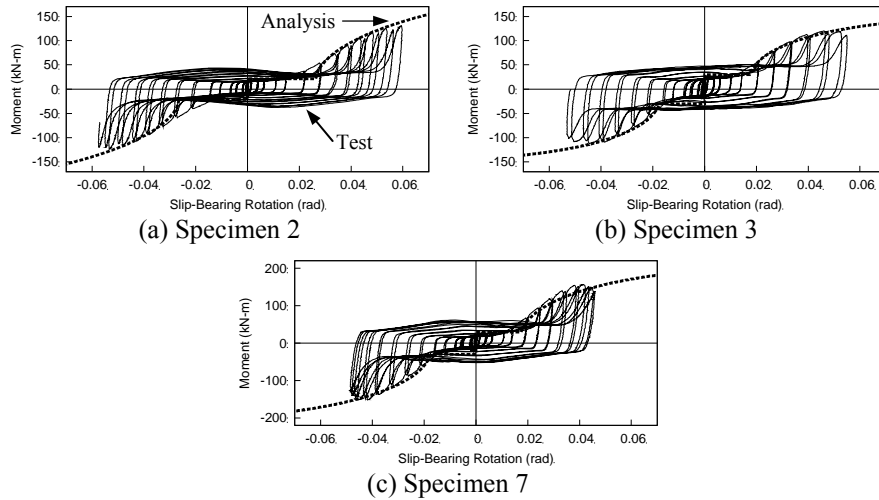


FIGURE 9 – Correlation of Response Envelope

### CYCLIC LOADING ANALYSIS

For a given bolt configuration and story height, the slip range shown in Figure 9 under monotonic loading is a function of the bolt hole oversize. But the cyclic test results also showed that the slip range would increase with the story drift. This resulted from the elongation of the bolt hole due to prior bearing deformation. For cyclic modeling, therefore, the effect of hole ovalization needs to be considered. Referring to Figure 8, the value of hole oversize ( $h_{OS}$ ), with a proper consideration of the relative bearing strength between the beam and column webs [Sato and Uang, 2008], needs to be updated in the cyclic analysis. Rigid unloading is assumed.

Figure 10 shows the cyclic correlation for three representative specimens. Note that the growth of slip range was reasonably simulated in the proposed model.

### SUMMARY AND CONCLUSIONS

As part of the AISI's ongoing effort to develop a standard for the seismic design of cold-formed steel structures (AISI S110), cyclic testing of nine full-scale beam-column subassemblies was conducted. These subassemblies represented a portion of the Cold-Formed Steel—Special Bolted Moment Frames (CFS—SBMF) which are commonly used in industrial platforms. This type of frames is generally composed of cold-formed HSS columns and double-channel beams interconnected

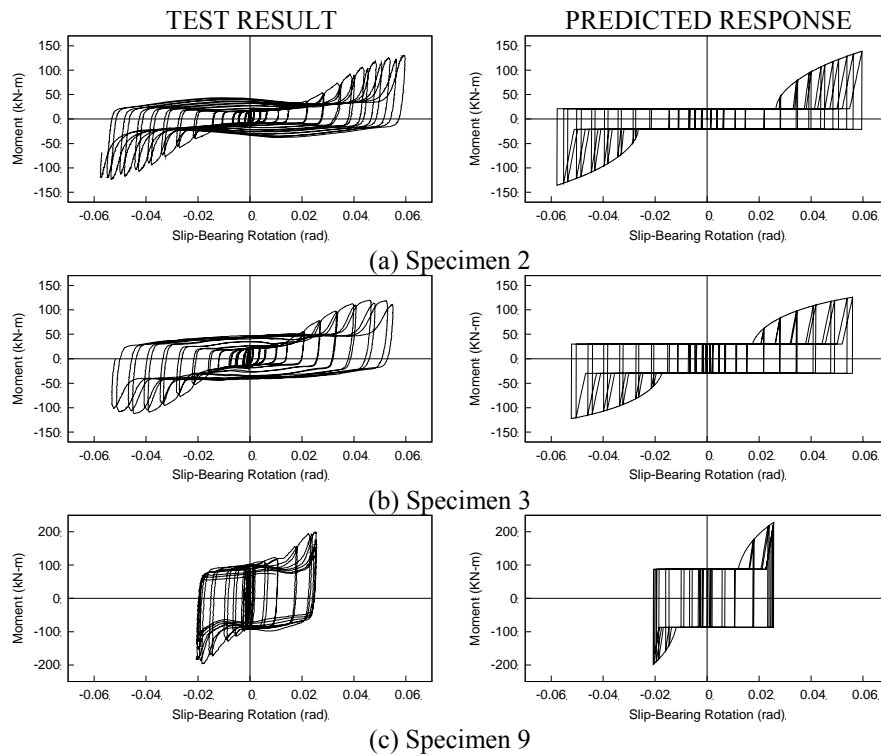


FIGURE 10 – Correlation of Cyclic Response

by snug-tight high-strength bolts. Specimens were designed such that the response of three failure modes—connection failure, beam buckling, and column buckling—could be studied. The following conclusions can be made.

- (1) All specimens were able to deform beyond 4% story drift in a ductile manner.
- (2) Typical response is characterized by three zones. Initially, these specimens responded elastically and the bolted connection acted like a rigid joint. A slip range then resulted, which corresponded to the response when the bolt friction was overcome. Bolt bearing in addition to friction then produced a region of significant hardening in strength until the specimen failed.
- (3) The bolt group in the connection region was subjected to an eccentric shear from the base of the column. All specimens showed ductile behavior due to this action; this desirable limit state involved bolt friction and bearing.

- (4) Beam local buckling was most undesirable and should be avoided by capacity design as it resulted in a significant degradation in strength. Although such local buckling occurred at a story drift beyond 4%, it is prudent to limit the flat width-to-thickness ratio of the beam web to  $6.18\sqrt{E/F_y}$  to control web local buckling.
- (5) Local buckling in HSS columns, which involved buckling of stiffened elements, could also result in a significant strength degradation. To avoid such strength degradation, test results showed that it is desirable to limit the flat width-to-thickness ratio to  $1.58\sqrt{E/F_y}$ .
- (6) A model which extends the instantaneous center of rotation concept of an eccentrically loaded bolt group for the simulation of cyclic behavior of the bolted moment connections was proposed. Considering both the friction and bearing resistance mechanisms as well as the bolt hole oversize, the simulated cyclic response correlated well with the test results.

#### ACKNOWLEDGMENT

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