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Strength of Concrete-Filled Steel Box Columns with Local Buckling Effects

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1.   Introduction

Concrete-filled steel box columns offer excellent structural performance, such as high strength, high ductility and large energy absorption capacity and have been widely used as primary axial load-carrying members in high rise buildings, bridges and offshore structures. Figure 1 shows the cross-sections of concrete-filled steel box columns. Local buckling of concrete-filled thin-walled steel box columns under axial compression is characterized by the outward buckling mode of the steel box. The restraint provided by the concrete core increases the critical local buckling stress of the steel box. On the other hand, the steel box completely encases the concrete core so that the ductility of the encased concrete can be improved. The steel box also serves as longitudinal reinforcement and permanent formwork for the concrete core, which results in significant savings in materials and labor costs.

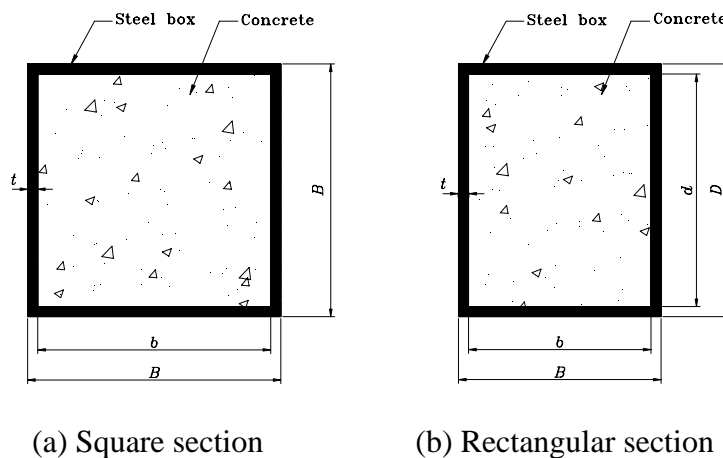(a) Square section   (b) Rectangular section

Figure 1 Concrete-filled steel box columns

Tests on concrete-filled steel tubular columns have been undertaken by many researchers. Furlong (1967) conducted tests on the ultimate loads of concrete-filled steel box columns and found that the axial load was resisted independently by the steel and concrete components and was not affected by concrete confinement. Knowles and Park (1969) studied the experimental behavior of circular and square concrete-filed steel tubular columns. Their results indicated that the circular steel tube offered confinement to the concrete core and the confinement increased the ultimate loads of short concrete-filled steel tubular columns. No confinement effect on the ultimate loads, however, was observed in concrete-filled square and rectangular steel box columns. Moreover, Tomii et al. (1977) investigated the effects of cross-sectional shapes on the concrete confinement of concrete-filled steel tubular columns. Furthermore, Shakir-Khalil and Mouli (1990) and Schneider (1998) have conducted tests on the experimental ultimate loads and behavior of concrete-filled steel tubular columns.

The ultimate loads and behavior of concrete-filled thin-walled steel box columns are influenced by the local buckling of the steel box walls. Ge and Usami (1992) presented experimental studies on the local buckling of concrete-filled steel box columns with and without internal stiffeners. Wright (1995) used an energy method to derive limiting width-to-thickness ratios for proportioning thin steel plates in contact with concrete. Experimental studies on the ultimate strengths of concrete-filled steel box columns with local buckling effects have been conducted by Uy and Bradford (1995), Bridge et al. (1995) and Uy (2000). Moreover, Liang and Uy (1998, 2000) proposed effective width models for the analysis and design of steel plates in concrete-filled thin-walled steel box columns. Furthermore, Liang et al. (2003, 2004) proposed buckling and ultimate strength interaction equations for the design of steel plates in double skin composite panels under biaxial compression and shear.

Nonlinear analysis methods for predicting the inelastic behavior of steel-concrete composite columns have been reported in the literature. El-Tawil et al. (1995) presented a fiber element analysis method for modeling the inelastic behavior of concrete-encased composite columns under axial load and biaxial bending. El-Tawil and Deierlein (1999) investigated the strength and ductility of concrete-encased composite columns using the nonlinear fiber element analysis. Lakshmi and Shanmugam (2002) presented a semi-analytical method for predicting the behavior of concrete-filled steel box columns. The current state of the art of nonlinear analysis of steel-concrete composite structures was reviewed by Spacone and El-Tawil (2004). The effects of local buckling of steel plates in concrete-filled steel tubular columns, however, are not considered in most nonlinear analysis methods that lead to the overestimates of the ultimate loads of composite columns and frames (Liew et al. 2001).

This paper presents a nonlinear fiber element analysis method for predicting the strength and behavior of concrete-filled thin-walled steel box columns with local buckling effects. By adopting the effective width models, the effects of local buckling on the strength and behavior of composite columns are taken into account in the nonlinear fiber element analysis. The progressive local and post-local buckling is simulated by gradually redistributing stresses within the steel box. The accuracy of the fiber element analysis method developed is established by comparisons with experimental results.

2.   Fiber element discretization

In the fiber element analysis, the composite section is discretized into many small regions (fibers), as shown in Figure 2. In the present fiber element program, the steel box wall is divided into m_s layers through its thickness and the discretization of fibers along the width of the wall is automatically undertaken based on the layer size of the wall. The concrete core is divided into m_c fibers in the x direction and the size of fibers in the y direction is automatically adjusted according to the size of fibers in the x direction. The discretization of the cross section results in square fiber elements. Steel fibers are grouped together as well as concrete fibers.

3.   Material models for structural steel

In the fiber element analysis, it is assumed that steel and concrete fibers in a composite section under axial loads are subjected to the same longitudinal strain. The constitutive models are based on the uniaxial stress-strain relationships of materials. The steel section can be made of mild steel or high strength and cold-formed steels. For mild structural steels, an idealized trilinear stress-strain relationship is employed in the fiber element analysis (Liang et al. 2004). For high strength and cold
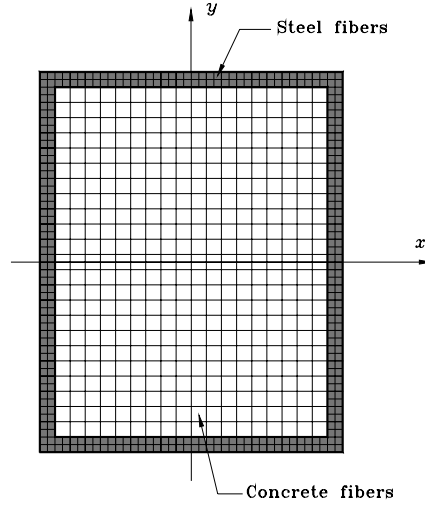
Figure 2 Fiber element discretization

formed steels, the stress-strain behavior is characterized by a rounded stress-strain curve. The material model suggested by Ramberg-Osgood (1943) is adopted in the fiber element program to calculate fiber stresses for high strength and cold formed steels. The Ramberg-Osgood formula is expressed by

$$\varepsilon_s = \frac{\sigma_s}{E_s}\left[1 + \frac{3}{7}\left(\frac{\sigma_s}{\sigma_{0.7}}\right)^n\right]$$

(1)

where σ_s is the longitudinal stress in steel, ε_s is the longitudinal strain in steel, E_s is the Young's modulus of steel, $\sigma_{0.7}$ is the stress corresponding to $E_{0.7} = 0.7E_s$, and n is the knee factor that defines the sharpness of the stress-strain curve. The knee factor $n = 25$ is used in the fiber element analysis program to account for the isotropic strain hardening of steel sections (Liang and Uy 2000). For given fiber strains, fiber stresses are determined from Eq. (1) using a numerical procedure.

4.   Material models for concrete

It is assumed that the confinement effect increases only the ductility of the concrete in concrete-filled steel box columns but not its strength (Tomii and Sakino 1979). The general stress-strain curve for concrete in concrete-filled steel box columns is depicted in Figure 3. The part OA of the stress-strain curve is modeled using the equation suggested by Mander et al. (1988) as

$$\sigma_c = \frac{f_c' \gamma (\varepsilon_c / \varepsilon_c')}{\gamma - 1 + (\varepsilon_c / \varepsilon_c')^\gamma}$$

(2)

where σ_c is the longitudinal compressive concrete stress, f_c' is the compressive cylinder strength of concrete, ε_c is the longitudinal compressive concrete strain, ε_c' is the strain at f_c'. The parameter γ is determined by

$$\gamma = \frac{E_c}{E_c - (f_c' / \varepsilon_c')}$$

(3)

3

where E_c is the Young's modulus of concrete, which is given by (ACI-318, 2002)

$$E_c = 3320\sqrt{f_c^{'}} + 6900 \quad (\text{MPa}) \tag{4}$$

The strain $\varepsilon_c^{'}$ is taken as 0.002 for concrete strength under 28 MPa and 0.003 for concrete strength over 82 MPa. When the concrete strength is between 28 and 82 MPa, the strain $\varepsilon_c^{'}$ is determined as a linear function of the concrete strength.
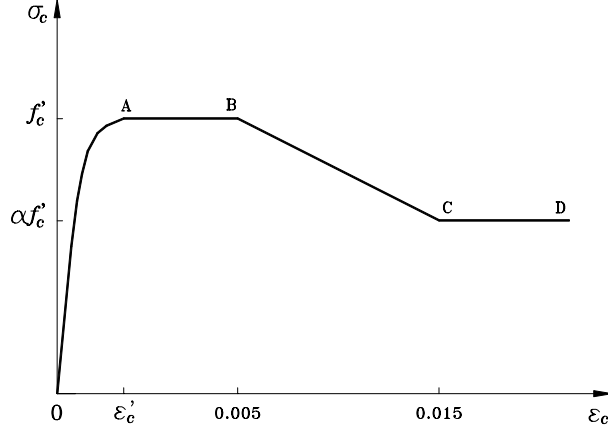

Figure 3 General stress-strain curve for concrete in concrete-filled steel box columns

The parts AB, BC, CD of the stress-strain curve for confined concrete are defined as follows:

$$\sigma_c = f_c^{'} \quad \text{for } \varepsilon_c^{'} < \varepsilon_c \leq 0.005 \tag{5}$$

$$\sigma_c = \alpha f_c^{'} + 100(0.015 - \varepsilon_c)(f_c^{'} - \alpha f_c^{'}) \quad \text{for } 0.005 < \varepsilon_c \leq 0.015 \tag{6}$$

$$\sigma_c = \alpha f_c^{'} \quad \text{for } \varepsilon_c > 0.015 \tag{7}$$

where α is taken as 1.0 when the width-to-thickness ratio (*B/t*) of the composite column is less than 24 and is taken as 0.0 when the *B/t* ratio is greater than 64 (Tomii and Sakino 1979). For *B/t* ratios between 24 and 64, α is taken as 0.6 in the present fiber element program.

5.   Critical local buckling

Local buckling reduces the strength and stiffness of concrete-filled thin-walled steel box columns and must be accounted for in the nonlinear analysis methods. Local buckling of steel plates is influenced by the width-to-thickness ratios, boundary conditions, initial geometric imperfections and residual stresses induced by welding or cold-formed process (Liang and Uy 2000). For perfect steel plates, the critical elastic buckling stress can be determined by the following equation (Bulson 1970)

$$\sigma_{cr} = \frac{k\pi^2 E_s}{12(1 - v^2)(b/t)^2} \tag{8}$$

4

where b is the width of the plate, t is the thickness of the plate, ν is the Poisson's ratio and k is the elastic buckling coefficient. The minimum elastic local buckling coefficient of 9.81 is used in Eq. (8) in the fiber element program for steel plates in concrete-filled thin-walled steel box columns as suggested by Liang (2005).

The studies conducted by Liang and Uy (2000) show that the critical local buckling stress (σ_{cb}) of steel plates with initial geometric imperfections and residual stresses is much less than that of perfect plates. In the proposed fiber element analysis method, the critical local buckling stresses of thin steel plates with the initial out-of-plane deflection of $0.1t$ and residual compressive stress of $0.25 f_y$ are approximately evaluated based on the results obtained from the nonlinear finite element analyses by Liang and Uy (2000). It is assumed that very stocky steel plates can attain the full plastic strength without local buckling effects.

6.   Post-local buckling

Post-local buckling is characterized by the stress redistribution within the buckled steel plate under axial compression. The effective width concept can be used to express the post-local buckling strength of thin steel plates as illustrated in Figure 4. This concept assumes that at the ultimate state, effective steel fibers are stressed to the yield strength of the steel material while the stresses of ineffective steel fibers are zero. Effective width formulas proposed by Liang and Uy (2000) are employed in the proposed fiber element analysis program and are expressed by

$$\frac{b_e}{b} = 0.675\left(\frac{\sigma_{cr}}{f_y}\right)^{1/3} \quad \text{for } \sigma_{cr} \leq f_y \tag{9}$$

$$\frac{b_e}{b} = 0.915\left(\frac{\sigma_{cr}}{\sigma_{cr} + f_y}\right)^{1/3} \quad \text{for } \sigma_{cr} > f_y \tag{10}$$

where b_e is the effective width of a steel plate and f_y is the yield strength of the steel plate. The above effective width formulas account for the initial out-of-plane defection of $0.1t$ and residual compressive stress of $0.25 f_y$.
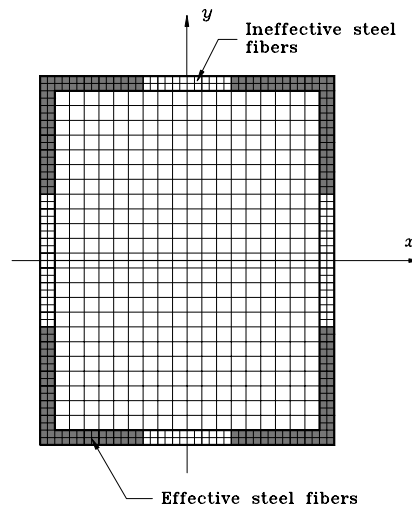

Figure 4 Effective width of steel plates in concrete-filled steel box columns

In the proposed fiber element analysis method, the progressive local and post-local buckling of steel plates in concrete-filled steel box columns is simulated by gradually redistributing stresses within the steel plates. After the critical local buckling, the ineffective width of a steel plate increases from zero to a maximum value when the applied load is increased to the ultimate load of the steel plate as shown in Figure 4. The maximum ineffective width of a steel plate at its ultimate load can be calculated by

$$b_{ne,\max} = b - b_e \tag{11}$$

The ineffective width of the steel plate between zero to $b_{ne,\max}$ can be approximately evaluated using linear interpolation based on the stress levels of steel fibers as

$$b_{ne} = \left(\frac{\sigma_s - \sigma_{cb}}{f_y - \sigma_{cb}} \right) b_{ne,\max} \tag{12}$$

where σ_{cb} is the critical local buckling stress of a steel plate with initial geometric imperfections and residual stresses. It is noted that the effective width determined by Eq. (9) or Eq. (10) is an ultimate strength criterion that governs the ultimate strength of a steel plate. The ultimate load calculated for the steel plate at any loading stage must not be greater than that determined using the effective width formulas. As a result, if $\sigma_s (b - b_{ne}) t > f_y (b - b_{ne,\max}) t$, the steel fiber stresses must be reduced using linear interpolation to satisfy the effective width criterion as

$$\sigma_s = \left(\frac{b - b_{ne,\max}}{b - b_{ne}} \right) f_y \tag{13}$$

In the fiber element analysis, fiber stresses are firstly calculated using material models from fiber strains. The fiber element analysis program then checks for the critical local buckling of steel plates. If steel fiber stresses are greater than the critical buckling stress, the effective width of the steel box walls is calculated and the stresses of steel fiber elements located within the ineffective width (b_{ne}) of the steel box walls are assigned to a zero value. Steel fiber stresses are updated to satisfy the effective width criterion. After the initial local buckling, the ineffective width grows with an increase in the applied load until it reaches the maximum value ($b_{ne,\max}$).

7. Stress resultants

The axial load applied to the composite column due to an axial strain deformation is determined as the stress resultant in the composite section, which is expressed by

$$P = \sum_{i=1}^{ns} \sigma_{s,i} A_{s,i} + \sum_{j=1}^{nc} \sigma_{c,j} A_{c,j} \tag{14}$$

where P is the axial load, $\sigma_{s,i}$ is the longitudinal stress at the centroid of steel fiber i, $A_{s,i}$ is the area of steel fiber i, $\sigma_{c,j}$ is longitudinal stress at the centroid of concrete fiber j, $A_{c,j}$ is the area of concrete fiber j, ns is the total number of steel fiber elements and nc is the total number of concrete fiber elements. The ultimate strength of a short concrete-filled steel box column is determined as the maximum load in the axial load-strain curve of the composite column.

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8.   Comparisons with experimental results

8.1   Load-axial strain curves

The axial load-strain curves of concrete-filled steel box columns predicted by the proposed fiber element analysis method are compared with experimental results presented by Schneider (1998) in this section. All steel boxes of the specimens were cold-formed carbon steel and were welded and annealed to relieve residual stresses. The Ramberg-Osgood material model was employed in the fiber element analysis for cold-formed steel sections. Experiments conducted by Cusson and Paultre (1994) indicated that the maximum compressive stress of concrete in columns varies from $0.85 f_c^{'}$ to $1.0 f_c^{'}$. This is due to the difference between concrete in a test cylinder and a column, the variation in the concrete compaction, water-cement ratio and curing conditions and the differences in loading rates between cylinder and column tests. In the present fiber element analysis, the maximum concrete compressive stress in the constitutive model was taken as $1.0 f_c^{'}$ for columns S1, S3, R1, R2 and R3 and $0.85 f_c^{'}$ for specimen S2. Figure 5 shows the comparisons of the axial load-strain curves predicted by the proposed fiber element analysis method with experimental results. It appears from Figure 5 that the proposed computational technique predicted very well the axial stiffness, ultimate strengths and post-peak behavior of the test specimens. The mean predicted ultimate load for all specimens is 97% of the experimental results. It can be concluded that the fiber element analysis program developed is capable of capturing the complete axial load-strain behavior of concrete-filled steel box columns with local buckling effects.

8.2   Ultimate strength

The ultimate strengths of concrete-filled steel box columns predicted by the proposed fiber element analysis method are compared with corresponding experimental results presented by Shakir-Khalil and Mouli (1990), Bridge et al. (1995) and Uy (1998) in Table 1. The load was applied to the steel plates only in the test of specimens B5-B29, NS5, NS11 and NS17. In the fiber element analysis, the maximum concrete compressive stress was taken as $0.85 f_c^{'}$ in the constitutive model. It can be seen from the table that the mean ultimate strength of all specimens predicted using the fiber element analysis program is 95.6% of the experimental value. It can be concluded that the proposed fiber element analysis method is reliable and conservative in predicting the ultimate strengths of concrete-filled thin-walled steel box columns with local buckling effects.

9.   Conclusions

A nonlinear fiber element analysis method has been presented in this paper for predicting the ultimate strengths and behavior of short concrete-filled thin-walled steel box columns with local buckling effects. The confinement effects on the ductility of the encased concrete in concrete-filled steel box columns are considered in the method. Effective width models proposed for steel plates in concrete-filled steel box columns with geometric imperfections and residual stresses are incorporated in the fiber element analysis method to account for local bucking effects. The fiber element analysis program simulates the progressive local and post-local buckling by gradually redistributing stresses within the steel box. It is demonstrated that the fiber element analysis program developed predicts well the ultimate strengths and behavior of concrete-filled steel box columns with local buckling effects. The fiber element analysis method presented can also be employed in the advanced analysis of composite frames.
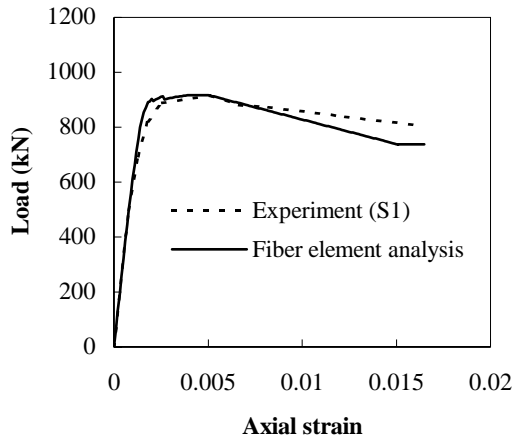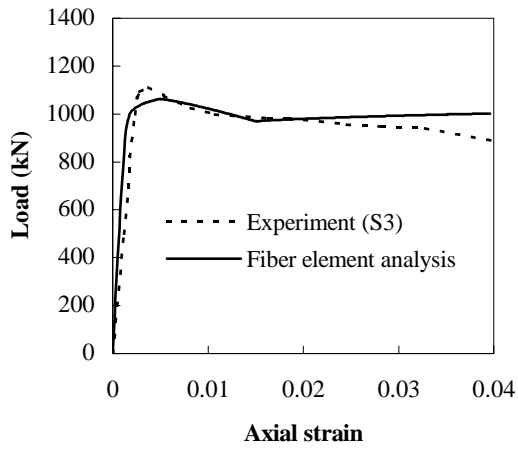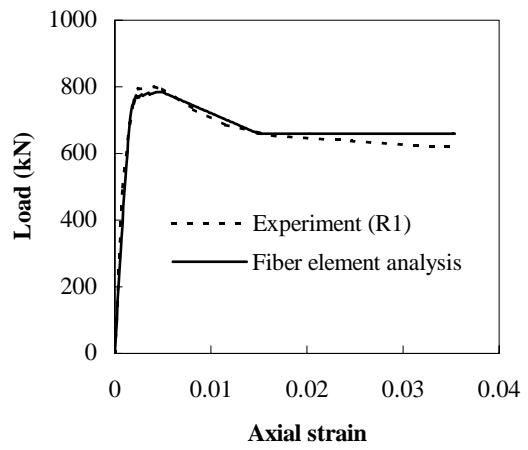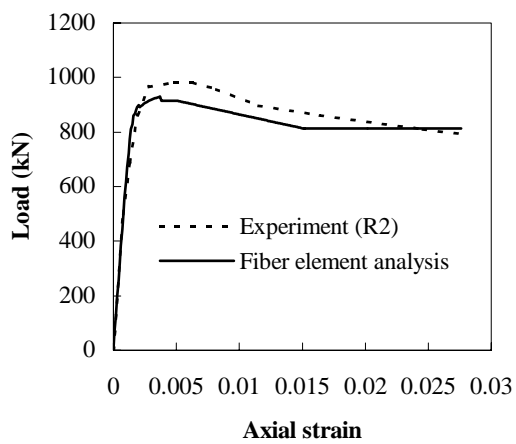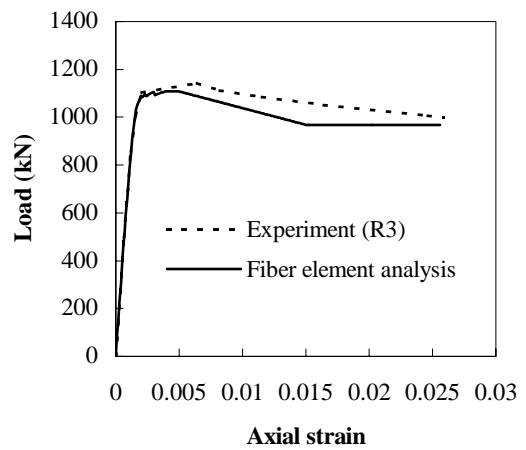
(a) Specimen S1
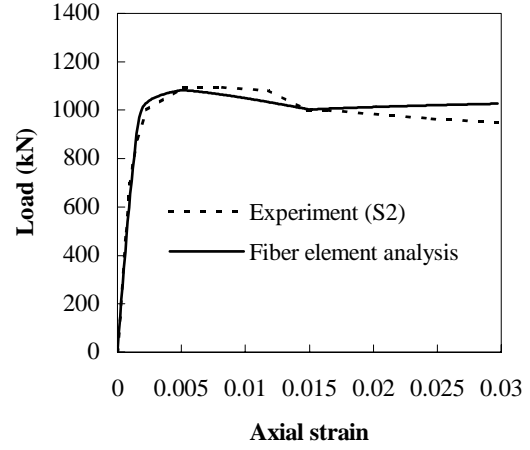
(b) Specimen S2

(c) Specimen S3

(d) Specimen R1

(e) Specimen R2

(f) Specimen R3

Figure 5 Comparisons of fiber element analysis with experimental results

Table 1 Comparisons of fiber element analysis with experimental results

Specimen	$B{\times}D$ (mm)	t (mm)	f_y (MPa)	E_s (GPa)	f_c' (MPa)	$P_{u.fib}$ (kN)	$P_{u.exp}$ (kN)	$\dfrac{P_{u.fib}}{P_{u.exp}}$
B29	282.14×282.14	2.14	282	199.4	-	332	364.2	0.912
B5	202.14×202.14	2.14	282	199.4	-	295.9	311.8	0.949
B20	162.14×162.14	2.14	282	199.4	-	290.3	269	1.079
B17	122.14×122.14	2.14	282	199.4	-	233.8	265.5	0.881
B16	82.14×82.14	2.14	282	199.4	-	167	185	0.903
NS5	186×186	3.0	281	200	-	485.6	517	0.939
NS11	246×246	3.0	292	200	-	567.6	563	1.008
NS17	306×306	3.0	281	200	-	596.8	622.3	0.959
NS1	186×186	3.0	294	200	33.6	1433.4	1555	0.922
NS7	246×246	3.0	292	200	40.6	2555.3	3095	0.826
NS13	306×306	3.0	281	200	44	3962.3	4003	0.99
NS14	306×306	3.0	281	200	47	4192	4253	0.986
NS15	306×306	3.0	281	200	47	4192	4495	0.933
NS16	306×306	3.0	281	200	47	4192	4658	0.90
C1	120×80	5.0	357.5	205	35.7	916.4	850	1.078
C2	120×80	5.0	341	205	38.8	905.8	900	1.006
C3	120×80	5.0	341	205	40.5	916.9	920	0.997
C4	120×80	5.0	362.5	205	39.1	948	950	0.998
C5	120×80	5.0	362.5	205	36	927.7	955	0.971
C6	150×100	5.0	346.7	209.6	38.5	1251.9	1370	0.914
C7	150×100	5.0	346.7	209.6	38.3	1249.7	1340	0.933
C8	150×100	5.0	340	208.6	38.7	1238	1300	0.952
C9	150×100	5.0	340	208.6	39.6	1247.7	1320	0.945
Mean								0.956

10.   Acknowledgements

11.   References

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