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2 **Crack Width Analyses of 50-Story Reinforced Concrete Building by**
3 **Parallelized Discrete-Like Crack Model**

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9
10 **ABSTRACT**

11 The discrete-like crack model (DLCM), a crack-analysis scheme incorporated with nonlinear
12 finite element method, is redeveloped for parallelism. The algorithm of the new DLCM based on
13 the theory of bond redistribution is implemented through the OpenMP interface and enables
14 analyses of a 50-story building with more than four million degrees of freedom. The analysis
15 considers the drying shrinkage and creep during the construction and service periods, and static
16 pushover loading and a time history of a near-fault strong ground motion. The amount of cracks is
17 quantified as the total length that occurs on the concrete surfaces. The structural damage due to
18 compression softening exhibits a strong correlation with the length of cracks with opening widths
19 larger than 2 mm. The analyzed building becomes unrepairable when the total length of the >2
20 mm-wide cracks exceeds 50 m. However, the occurrence of such a serious damage induced by any
21 seismic vibrations ever recorded is unlikely.

22 **PRACTICAL APPLICATIONS**

23 “There’s no art to find the mind’s construction in the face,” so King Duncan said. However,
24 we can consider the search for structural damages on concrete surfaces a form of art. The presence
25 of cracks proves the life of a reinforced concrete building, similar to a person’s history engraved

as wrinkles and scars in their face. Our novel parallel numerical scheme, namely, discrete-like crack model (DLCM), traces not only the long history of a building starting from its construction but also resolves the issue of unexpected near-fault pulse-type earthquakes into detailed profiles. In this case study, DLCM is employed to assess structural damage using a percentage of cracks that extend up to hundred kilometers on the entire concrete surfaces of a 50-story residential building. The profiles of cracks, stresses, and strains articulate the building's past and current status and render possibility to improve a quality of living in the future.

INTRODUCTION

The developed discrete-like crack model (DLCM) (Sato 2007; Sato 2014) considers the bond stress between concrete and steel, and its incorporation into the finite element method (FEM) enables a performance-based design (PEER/TBI 2010) through as-is visualization of damages in reinforced concrete (RC). The recent progress in FEM demonstrates the influence of drying shrinkage cracks on seismic performance (Kurihara 2017; Kitazawa 2018). In addition, the effects of near-fault ground motion, which have been recorded since the late twentieth century, have induced unexpected damages (Kawai 2021). Lifecycle assessment is also expected by considering the shrinkage and creep starting from construction (Tahmasebinia 2019). This kind of assessment is especially demanded for thick shear-resistant walls of high-rise RC buildings because of the difficulties of counter-earthquake retrofitting and postearthquake repair compared with those of thinner members, such as columns and beams. Therefore, large-scale simulations have been performed to detect unexpected local damage in the overall structural behavior by means of parallel computing (Cho 2012; Cho 2013; Fu 2015; He 2016; Ohsaki 2009).

The flow in FEM is classified into (a) element-state determination (ESD) and (b) solution to simultaneous linear equations (SLE). The latter was formerly considered the most time-consuming part, but the former is currently the focus of research because of the strong nonlinearity of up-to-date computational tasks. Nonlinear phenomena naturally include order-dependent advance/retreat

processes, which are usually integrated in models with an order-independent process. The developer often fail to recognize the boundaries between these two kinds of processes. Parallelization requires the detection of boundaries and complete separation. The domain decomposition method (DDM) is one of the most successful parallelism methods (Farhat 1988; Yamagawa 1991; Yamagawa 1993; Chan 1994), and it has been incorporated with hybrid interfaces of the message-passing interface and OpenMP (Ohsaki 2009; Nikolić 2015). The former demonstrates high performance, and the latter flexibly shares workload (Armstrong 2000; Jost 2003; Fu 2015; He 2016). Recent studies (Oh 2017; Lee 2017; Couturier 2000; Amritkar 2012) preferred the use of OpenMP because the source code can be switched between sequential and parallel operations without modifications. DLCM repeats the same subroutines in the parallel and sequential calculations; thus, OpenMP is significantly affinitive.

Among the methods used to calculate crack widths in RC (Ngo 1968, Wijerathne 2009; Etse 2012; Kim 2008; Yamamoto 2014; Le 2016; Khmurovska 2017), DLCM is fitted to the FEM based on the smeared-crack model (Rashid 1968). The old DLCM (Sato 2007; Sato 2014; Akai 2017; Kitazawa 2018) contains order dependence that hinders parallelization. The present study redevelops DLCM and performs analyses that chronologically cover the lifecycle of a 50-story RC building from the construction stage until seismic vibrations.

DLCM AND PARALLELIZATION

The DLCM applies solutions of ordinary differential equations to evaluate the development length l_d in a smeared-crack field (Kaufmann 1998). When the reinforcement is elastic, the equation is expressed by Eqs. (1) and (2) (Arnovljević 1909; Rehm 1961; Edwards 1972).

$$\frac{d^2S}{dx^2} = \frac{S_1}{\tau_{b1}} \cdot \beta^2 \cdot \tau_b \dots\dots\dots (1)$$

$$\beta = \sqrt{\frac{4 \cdot \tau_{b1}}{d_b \cdot S_1} \left(\frac{1}{E_s} + \frac{\rho_s}{E_c} \right)} \dots\dots\dots (2)$$

where d_b is the bar diameter, E_c and E_s are the elastic moduli of the concrete and reinforcement, respectively, ρ_s is the reinforcement ratio, τ_{b1} and S_1 are the characteristic bond stress and slip, respectively, and x is the coordinate along l_d as shown in Fig. 1(b) (Eligehausen 1983; fib 2010; Sato 2014). When the reinforcement yields, β is replaced by β_p , which is given by Eq. (3).

$$\beta_p = \sqrt{\frac{4 \cdot \tau_{b1}}{d_b \cdot S_1} \left(\frac{1}{E_{sh}} + \frac{\rho_s}{E_c} \right)} \quad (3)$$

where E_{sh} is the reinforcement strain-hardening modulus. When the reinforcement yields and the bond stress–slip relationship is on the unloading/reloading paths, the differential equation is expressed by Eq. (4) using bond stress τ_{bu} at the start of unloading (Sato 2014).

$$\frac{d^2 S}{dx^2} = \frac{S_1}{\tau_{b1}} \cdot \{(\beta_p^2 - \beta^2) \cdot \tau_{bu} + \beta^2 \cdot \tau_b\} \quad (4)$$

Equation (5) yields released stress $\Delta\sigma_{cb}$, which is subtracted from the current concrete stress.

$$\Delta\sigma_{cb} = \frac{4\rho_s}{d_b} \left(\int_0^x \tau_b \cdot dx - \int_0^{l_d} \tau_b \cdot dx \right) (< 0) \quad (5)$$

The compatibility of reinforcement is satisfied at both ends of the l_d , whereas incompatibility is allowed within l_d . This assumption enables redistribution without additional degrees of freedom. Figures 1(d) and 1(e) display the crack-detection process. The gray areas in Fig. 1(d) reveal the l_d of the first and second cracks. Stresses are redistributed [Fig. 1(e)] by $\Delta\sigma_{cb}$ [Eq. (5)], and elements within the l_d are excluded. New cracks are initiated by the existing ones in adjacent elements. The concentration stress induced by adjacent cracks is given by Eq. (6) (Sato 2007):

$$\Delta\sigma_{ce} = 0.5\sigma_{cr} \quad (6)$$

Equation (7) gives new stress σ_{cnew} as a summation of previous stresses σ_{cold} , $\Delta\sigma_{cb}$, and $\Delta\sigma_{ce}$:

$$\sigma_{cnew} = \sigma_{cold} + \Delta\sigma_{cb} + \Delta\sigma_{ce} \quad (7)$$

A new crack can occur only when σ_{cnew} exceeds the cracking stress of concrete σ_{cr} . The centroid of the second crack is continuously aligned with the first crack [Fig. 1(f)].

Figure 1(g) shows the flow of conventional FEM. In a nonparallel computation, the algorithm performs ESD in C_1 and solves SLE in D_1 . When incorporated with DDM, the former and latter decompose from C_1 to C_n and from D_1 to D_n , respectively, which result in a speed-up.

Figure 1(h) shows the parallelism of DLCM. The ESD (i.e., A_1 , B_1 , and C_1) needs longer times than SLE in D_1 because DLCM calculates $\Delta\sigma_{cb}$ and $\Delta\sigma_{ce}$ in A_1 , redistributes the new crack-related stresses in B_1 , and estimates the stiffness in C_1 . The serious problem of the old DLCM was the order dependence contained in Eq. (7). A_1 traces cracking [Eqs. (1) and (4)–(7)] through a chess-game-like advance/retreat process, where element stress σ_{cnew} continuously switches between positive and negative values. The new DLCM subdivides A_1 into the following: (1) A_a is used for the exclusive calculation of the maximum released stresses, (2) A_b for the calculation of the minimum concentration stresses, and (3) A_c for the computation of resultant stresses. Exclusive extraction of negative values in A_a and positive values in A_b and the simple summation in A_c eliminate the order dependence and demonstrate a strong affinity to parallelism.

The new crack-related block B_1 cannot be parallelized. Meanwhile, parts C_1 and D_1 are parallelized in the same manner as in the DDM. The times of C_1 and D_1 are shorter than those of conventional FEM because the contributions of A_1 and B_1 allow relatively coarse meshes.

Figure 2 shows a comparison between the old and new DLCMs. Six existing cracks are present (thick line) among the total of 72 FEs, and several new cracks are expected (thin dotted lines). Loop A_1 of the old DLCM rigidly integrates the computations of $\Delta\sigma_{cb}$, $\Delta\sigma_{ce}$, and σ_{cnew} [Eq. (7)], which results in a long CPU time and order dependence. The new DLCM separates old A_1 into three new independent parallel loops. Loop A_{ai} redistributes the stresses along the reinforcement using Eq. (5). This process exclusively extracts the maximum released stress $\Delta\sigma_{cbmin}$. Loop A_{bi} detects the extended cracks. Parallel loops A_{ci} compute the new stress σ_{cnew} .

$$\sigma_{cnew} = \sigma_{cold} + \Delta\sigma_{cbmin} + \Delta\sigma_{cemax} \dots\dots\dots(8)$$

Estimation of element stiffnesses is then accomplished using the new parallel loop C_i instead of the old C_1 . The redeveloped DLCM maintains the nonlinear performance of the old version.

The algorithm, which spans 700 subroutines and more than 200,000 lines, is implemented in the FEM program FINAL (Naganuma 2004). SLEs are computed by MKL Pardiso (Intel 2022). The employed hardware has a CPU of 3.0GHz clock rate with 36 threads. A part of this work has been reported in a previous paper (Sato 2022); however, the static analysis of a 50-story building is expanded to dynamic and lifespan analyses using thrice degrees of freedom.

FIFTY-STORY BUILDING MODEL

The analyzed 50-story RC flat-slab model building (JICE 1993) has a height of 151.5 m and a 42.75 m-square plan. It is composed of 64 columns, four L-shaped walls connected with beams, and 250 mm-thick flat slabs, as shown in Fig. 3(a). Concrete strength f_c ranges from 60 to 100 N/mm². The 685-N/mm²-class steel is used for longitudinal reinforcement of the columns, walls, and beams, 1275-N/mm²-class steel is used for shear reinforcement of the columns and beams, and 980-N/mm²-class steel is used for lateral reinforcement of the walls. These high-strength steels guarantee the preyield state of columns and walls while allowing the plastic hinges on flat slabs to attain a total drift of 1.25%. The square cross sections of columns D_c range from 800 to 950 mm. The longitudinal bar ratios, i.e., ρ_l , range from 0.50% to 0.74%, and the shear reinforcement ratios, i.e., ρ_w , range from 0.36% to 0.53%. The wall-thickness values, i.e., t_w , range from 750 to 950 mm. The vertical bar ratios, i.e., ρ_g , are from 3.27% to 4.23%, and the ρ_w values are from 1.12% to 0.68%. The design is aimed at (1) limiting the crack width within 0.2 mm under normal service conditions for aesthetic considerations, (2) limiting the story drift within 1.25% under a seismic velocity of 500 mm/s, and (3) limiting the columns and walls within the peak strength in an ultimate state. The construction completes a story every week, in which the columns and walls are cast at the first day and the beams and slabs at the fifth day. The work spans 50 weeks until completion. Eight-node hexahedral elements are used for the columns and walls and four-node quadrilateral shell elements for the slabs. The column and wall of the first story are divided into $6 \times 6 \times 14$ and $6 \times 8 \times 14$ meshes, respectively, and those of upper stories into $6 \times 6 \times 11$ and $6 \times 8 \times 11$ meshes. All reinforcing steels are embedded in these concrete elements. The material

constitutive models are identical to those used in the previous studies [Fig. 1(a)] (Naganuma 2004; Willam-Warnke 1975; Ciampi 1982; fib 2010; Sato 2014). An improved model consisting of nonorthogonal crack coordinates and nonlinear shear transfer paths (Naganuma 2004) are also incorporated [Fig. 2(c)]. Geometric nonlinearity is considered. The degrees of freedom are 4,300,186, which are impossible to achieve using the old DLCM. The building is then subjected to analyses, which are chronologically classified into three stages.

(1) Fifty-week construction period: The construction process is modeled by removing the element stiffness of the unconstructed stories while drying-shrinkage strains, creep strains, and material-development ratios for tensile strength f_t , compressive strength f_c , and elastic modulus E_c are input [Fig. 3(b)]. Dead and live loads are applied to the constructed stories. Three steps per week are set, and 150 analysis steps are provided for the 50-week construction period. The shrinkage and creep strains are modeled based on previous studies (Sato 2005; Hagihara 2000; Hagihara 2002; Hagihara 2003; Kato 2019). In the past research, the creep coefficient given based on the double-power law developed by Bažant (Bažant 1976) is expressed as Eq. (9):

$$J(t - t_o) = 1 + 0.9t_o^{-0.33}(t - t_o)^{0.125} \dots\dots\dots(9)$$

where t refers to the material day, and t_o is the day at loading. The creep strain at every analysis step since loading accumulates based on the law of superposition [Fig. 3(b)].

The multilinear bond stress–slip models shown in Fig. 1(b) are input. Figure 3(c) shows a sample bond between the 100-N/mm² concrete and 41.3-mm-diameter bar in the first story. Bond-strength reduction due to steel yielding is considered, as shown in Fig. 3(c), in which an example of steel with 0.02 strain is drawn. The curve is further modified by considering the influences of drying shrinkage and creep according to our studies (Hong 2015; Ida 2016). Figure 3(d) shows the chronological change in the same bond curve for the 7th, 28th, and 1050th days.

(2) Two-year service period: Shrinkage and creep are applied to the completed building for three weeks in nine steps and for 100 weeks in 50 steps.

(3) Static pushover or 26-s seismic vibration: The building is then either statically pushed over or dynamically vibrated. In the former, prescribed horizontal displacement is applied using tournament-like trusses [Fig. 3(a)] up to a total drift (=roof displacement/building height) of 4% in 180 steps. The truss maintains force ratios that are determined in accordance with a primary eigenmode. The 50th story has a maximum force ratio of 5.27. In the latter, two types of 26-s 3D base accelerations are input in 2600 steps with a time increment of 0.01 s per step. These two types are the following: (i) TCU052 of 1999 Chi-Chi Earthquake ($PGA = 4965 \text{ mm/s}^2$, $PGV = 2334 \text{ mm/s}$) (VDC 2022) as an unexpected near-fault pulse-type motion and (ii) the type amplified by 280% as an ultimate excitation. Figure 3(e) shows the velocity history in the maximum direction (38° northeast) of the TCU052. TCU052 has been one of the most referred records since Mavroeidis initiated the study of near-fault strong motions (Mavroeidis 2003). It is selected because the 50-story building in a primary natural period of 3.26 s is expected to be sensitive to long dominant periods, as shown in the velocity spectrum (2.3 to 8.5 s) in Fig. 3(f), even though records of larger PGV (3,800 mm/s of TCU068) or larger PGA (12,660 mm/s^2 in San Fernando 1971) exist. Coefficients $\beta = 0.25$ and $\gamma = 0.5$ of the time increment are adopted for the Newmark β method. The damping is primary stiffness proportional type, and we add 1% damping in a rate of the primary natural period 3.26 s, which is evaluated by Subspace method.

The abovementioned three stages are continuously analyzed with no interruption so that the maximum crack widths of the lifespan can be evaluated. All static and dynamic analyses are classified into cases with and without the construction and service periods. Table 1 lists these six analysis cases. The total CPU times range from 63.8 h to 506.0 h (Table 1).

ANALYSIS RESULTS

Static Analyses: Cases 1 and 2

Figure 4(a) shows relationships between the base shear and total drift. The results indicate that the drying shrinkage and creep reduce the maximum base shear by 6.7%. The base shear achieves

a peak value at a total drift of 1.30% in Case 1 and 1.42% in Case 2. A concentrated story drift of 10% occurs at the 13th story for both cases. The maximum crack widths are 3.4 mm and 3.9 mm at the story drift of 1.25% and ultimately 113.9 mm and 85.6 mm, respectively.

Dynamic Analyses: Cases 3 to 6

Figures 4(b) and 4(c) display the time histories of the total drift. As shown in Table 1, the maximum base shears in Cases 4 and 6 are lower than their counterparts, i.e., Cases 3 and 5, respectively. The smaller base shear and reduced stiffness imply a possible increase in the drift. Cases 4 and 6, however, demonstrate reduced drifts compared with Cases 3 and 5, respectively, as presented in Table 1 and Figs. 4(b) and 4(c). This unforeseen phenomenon is attributed to two reasons. (i) Shrinkage and creep cause smooth stress redistributions, and (ii) the reduced stiffness extends the natural period and is insensitive to seismic excitations. An opposite phenomenon to the latter was observed in our previous study on a 22-story RC building (Kitazawa 2018) where the drifts increased due to resonance.

The maximum story drifts exceed 3.4% in Cases 5 and 6 with $\text{TCU052} \times 280\%$, which is roughly twice that in Cases 3 and 4 with $\text{TCU052} \times 100\%$. Among the four dynamic cases, Case 5 experiences the largest drifts, which stops midway of the vibration (12.4 s) due to numerical instability. The maximum crack widths (Table 1) indicate considerable openings of over 100 mm. The next section shows that Cases 3 and 4 are repairable while Cases 5 and 6 almost collapse.

Figure 5 displays Case 6 from the 50-week construction period through a two-year service period and up to a 26 s seismic vibration. Figure 5(b) shows the crack pattern halfway in the construction when the 25th story was built in the 25th week. Maximum crack widths of more than 0.2 mm develop in stories lower than the 20th story (gray solid line in Fig. 5(d)), where concrete with a strength greater than 80 N/mm^2 is used. At the end of the service period, widths over 0.2 mm can still be observed, as indicated by the dotted gray line in Fig. 5(d). Figure 6(a) shows the close-up cracks of the second-story floor slab, and Fig. 6(b) shows the center core of the first and

second stories at the end of the service period. Cracks that are 0.2-mm wide, which are indicated by the triangle symbols, are present. However, cracks measureless than a meter in length are located along the boundaries between slabs and columns and therefore may be unrecognizable to the inhabitants.

Figure 5(c) shows the cracks at the maximum drift due to vibration. The cracks become denser as the story becomes lower, and cracks of over 2-mm wide (circular symbols) appear in the stories lower than the 40th story. Several studies mentioned that the shear transfer along the crack began to disappear when the opening width exceeded 2 mm (Bažant 1984; Vecchio 2000; Muttoni 2008). In addition, the plastic elongations of the steels interfered with the closing of the more than 2-mm-wide cracks (Maeda 2004; Lantsoght 2016). The corresponding maximum crack widths are indicated by the black solid line in Fig. 5(d). The widths further increase at the final step, as shown by the dotted line. Figure 5(e) shows the entire cracking hysteresis from the 50-week construction period through a two-year service period and up to the 26-s seismic vibration in Case 6. The widths remain below 0.429 mm at the end of the service period. During the vibration, the maximum width of the 1st story attains 23.09 mm, whereas that of the 50th story is 0.89 mm.

Figure 5(a) illustrates the profiles of the strains, forces, and moment of Wall EF3 and Column H3 [Fig. 3(a)] of the selected ten stories on the seventh material day. The stories correspond to the locations where the wall and column configurations change. The dates are recorded two days after the casting of slabs and before the construction of the upper story. Thus, the walls and columns are restrained by the shrinkage of slabs while bearing no dead load except that of slabs. The circular symbols in the profiles denote the maximum values. The strains, vertical force, and moment are normalized by the cracking strain ε_{cr} , cracking axial force $D_c^2 \sigma_{cr}$, and cracking moment M_{cr} ($= D_c^3 \sigma_{cr} / 6$), respectively.

The normalized vertical ($\varepsilon_v / \varepsilon_{cr}$) and horizontal ($\varepsilon_h / \varepsilon_{cr}$) strains of Wall EF3 are 1.52 and 1.73, respectively, in the first story. The wall of the 100 N/mm² concrete with heavy orthogonal

reinforcement displays vertical and horizontal cracks. On the other hand, the normalized vertical tensile force $N/D_c^2/\sigma_{cr}$ drops to 0.21 cause of postcrack redistribution and the creep.

Compared with Wall EF3, Column H3 has a smaller maximum vertical strain ($\varepsilon_v/\varepsilon_{cr} = 1.07$) that results in a fewer horizontal cracks. Similar to that of the wall, the vertical tensile force $N/D_c^2/\sigma_{cr}$ drops to 0.31 because of redistribution and the creep. The normalized moment profile (M/M_{cr}), which remains smaller than 0.57, indicates that no cracking results from the column bending induced by the shrinkage of slabs. Therefore, the horizontal cracks observed in the column are caused by the axial tension of shrinkage.

A gap is discovered in the 35th and 36th stories of the M/M_{cr} of Column H3. This result is due to the relatively small cracking strain of the 60 N/mm² class concrete. On the other hand, the profiles are unaffected by the differences in the specific areas between the 35th and 36th stories.

Figures 4(d) and 4(e) show the profiles of Wall EF3 and Column H3 during the service period and seismic vibration of Case 6. Shear serves as an index of the horizontal resistance of Wall EF3. Fig. 4(d) shows the shear V normalized by $A_w\rho_w\sigma_{wy}$, where A_w , ρ_w , and σ_{wy} are the cross-sectional area, horizontal reinforcement ratio, and yield stress, respectively. For Column H3, horizontal resistance is indicated by the moment M normalized by the flexural capacity M_y . The four kinds of lines in Figs. 4(d) and 4(e) represent the profiles at the erection, end of service period (“2 year”), maximum seismic drift, and the final step. The wall’s shear V is 2.45 times of the $A_w\rho_w\sigma_{wy}$ during vibration, and the column’s moment remains at preyield values ($M/M_y = 0.49$). The shear and moment present low values during the service period and the final step. The axial force profiles (N) are hereby normalized using the compressive capacity $D_c^2f_c$ instead of the cracking strength $D_c^2\sigma_{cr}$. Their variations between the erection and end of service period, which are drawn by solid and dotted gray lines, are insignificant. At the maximum drift during seismic vibration, the normalized axial forces of Wall EF3 and Column H3 reach 0.20 and 0.14, respectively. The axial strain profiles (ε_v) are normalized using the strain corresponding to the compressive strength ε_c instead of the cracking strain ε_{cr} . The axial strains at stories higher than the 21st story of Wall EF3

drastically decrease by 97% during the service period, and those of Column H3 show insignificant change. The axial strain of Wall EF3 equals $2.68\varepsilon_c$ at the maximum seismic drift, and the value drops to almost zero at the final step. By contrast, the strain of Column H3 measures $1.91\varepsilon_c$ at the final step, and this value is twice that at the maximum drift ($0.95\varepsilon_c$). The steel bars of the column remain at preyield values, but the strain profile indicates crushing. The profiles denoted by “ R_E ” in Figs. 4(d) and 4(e) indicate the stiffness reduction of the concrete. Given the magnitude of reduction, a logarithmic index R_E is defined by Eq. (10):

$$R_E = \langle \sum_{n=1}^{N_{elm}} [\sum_{i=1}^3 \{1/\log_{10}(10E_c/E_i)\} + \sum_{j=1}^3 \{1/\log_{10}(10G_c/G_j)\}] / 6 \rangle / N_{elm} \cdots (10)$$

where E_i is the normal stiffness along the principal directions, G_c the elastic shear modulus; G_j the shear stiffness related to principal directions, and N_{elm} the number of elements, respectively. Shrinkage causes the R_E to drop to 0.7 in the top and bottom boundaries of the wall and column lower than the 10th story during the service period. $R_E = 0.7$ accounts for 37% of the elastic stiffness. The R_E further drops to 0.39 in Wall EF3 and 0.46 in Column H3 during seismic vibration, accounting for 2.8% and 6.4% of the elastic stiffness, respectively. These percentages roughly represent the ratios of lateral stiffness to the elastic state of Wall EF3 and Column H3.

Figures 6(c) and 6(d) compare final cracks of the center core walls in the first and second stories between Cases 4 and 6. Case 6 has more 2 mm-wide cracks (circular symbols) than Case 4. The presence of only 1% unsafe cracking among 99% of safe ones indicates the necessity of estimation of shrinkage cracks to extract critical cracks. Three ways are available to quantify the amount of cracks, i.e., using the (i) surface area, (ii) volume (= surface area multiplied by width), and (iii) crack length on the concrete surface. The third may be the most convenient method from the viewpoint of postearthquake assessment. Variables L_{cr} and $L_{cr_2\text{ mm}}$ are defined as the total length of the cracks and the more than 2-mm-wide cracks, respectively. The time histories of L_{cr} and $L_{cr_2\text{ mm}}$ in the six cases are shown in Figs. 4(a)–4(c) in units of kilometers. The L_{cr} values of Cases 2, 4, and 6 are larger than their counterparts, i.e., Cases 1, 3, and 5. The $L_{cr_2\text{ mm}}$ values of

Cases 1 and 3, however, become larger than those of Cases 2 and 4. The number of more than 2-mm-wide cracks is therefore not always proportional to the total number of cracks.

Damage Indices and Crack Width

The damage to concrete due to compression is not always as visible as that in cracks due to tension. The crack is a physically quite different phenomenon from compressive softening but has a close relationship in actual seismic vibrations. Quantification of the compressive deterioration was attempted in our previous study to represent the damage of an entire structure using single index $Di(c)$, which can be obtained by ordinary output data of any types of FEM software (Akai 2017). $Di(c)$ is expressed in Eqs. (11) to (13).

$$Di(c) = \{\sum_V (Di(c)_{elm} \cdot \varepsilon_{cmin}) dV\} / (\sum_V \varepsilon_{cmin} \cdot dV) \quad (11)$$

$$Di(c)_{elm} = W_{epc_elm} / W_{c0} \quad (12)$$

$$W_{epc_elm} = \sum_{k=1}^m (\sigma_{ij} \cdot d\varepsilon_{ij}) - \sigma_{ij} \cdot \varepsilon_{ij} / 2 \quad (13)$$

,where ε_{ij} represents the strain tensor of the finite element at each analysis step, $d\varepsilon_{ij}$ indicates the increment in ε_{ij} , k and m denote the current and final analysis steps, respectively, σ_{ij} corresponds to the stress tensor of the finite element at each analysis step, and W_{c0} is the plastic strain energy of the concrete defined in Fig. 6(e). $Di(c)$ is calculated in three steps. (i) The plastic strain energy of each element W_{epc_elm} is obtained by subtracting the elastic strain energy from the total strain energy, as expressed in Eq. (13). (ii) The damage index of each element $Di(c)_{elm}$ is obtained by normalizing W_{epc_elm} with W_{c0} . (iii) $Di(c)$ is calculated as an average $Di(c)_{elm}$ weighted with minimum principal strain ε_{cmin} . Figure 6(f) shows a sample correspondence between the cyclic stress–strain curve and $Di(c)_{elm}$. Figures 6(g) and 6(h) show the distributions of $Di(c)_{elm}$ at the center core walls in Cases 4 and 6, respectively. The damage of Case 4 is limited, whereas that of Case 6 drastically spreads up to the 40th story.

Figure 4(a) shows the relationship between the $Di(c)$ and the total drift of Cases 1 and 2. $Di(c)$ reaches 1.0 at drifts of 1.29% and 1.23%, which approximately coincide with the drifts of the peak base shear. The structure is therefore regarded as unrepairable when $Di(c)$ exceeds 1.0. Figures 4(b) and 4(c) show the time histories of $Di(c)$ in Cases 3–6. $Di(c)$ in Cases 5 and 6 exceeds 8.0, whereas that in Cases 3 and 4 remains under 1.0. Figures 6(i) and 6(j) show the comparison of the relationships between $Di(c)$, L_{cr} , and $L_{cr_2\text{ mm}}$ of the six cases. This comparison indicates that $Di(c)$ possesses a better correlation with $L_{cr_2\text{ mm}}$ than with L_{cr} . In the actual post-earthquake assessments, only the values of the unloaded states are available. Figure 6(k) therefore shows the extraction of the values at zero displacement during the vibrations in Cases 3–6. The curve fitting results in the square of the correlation coefficient of 0.96 and indicates that approximately, the total length of 50 m of the more than 2-mm-wide cracks corresponds to the unrepairable damage of the building. The inspection of the typical parts alone instead of the entire structure is more convenient. Figure 6(l) correlates the damage index with the crack area ($= \sum \{\text{crack length} \times \text{opening width}\}$) of Walls CD3 and EF3 in the first and second stories. The squared value of the correlation coefficient is 0.86, which balances the workload for inspection and the accuracy of damage prediction.

CONCLUSIONS

DLCM is redeveloped and parallelized through the OpenMP interface to evaluate the crack widths of a 50-story RC building. The new DLCM enables static and dynamic analyses of the FE model with more than four million degrees of freedom by taking into account the drying shrinkage and creep. On the basis of the analyses, the following findings are obtained.

(1) Cracks larger than 0.2 mm width are observed after the construction and service periods.

However, the amounts and locations are limited.

(2) The damage index of concrete $Di(c)$ is strongly correlated with the total length of the more than 2-mm-wide cracks, namely, $L_{cr_2\text{ mm}}$, whereas no correlation is found with L_{cr} .

(3) The building analyzed in this study is determined as unrepairable (i.e., $Di(c) > 1$) when $L_{cr_2\text{ mm}}$ exceeds 50 m.

Future research will be conducted to refine the DLCM via optimal parallelization and will include more case studies considering various conditions, such as soil–foundation interactions, to develop a damage threshold for RC buildings based on in-site observation of real crack patterns.

DATA AVAILABILITY STATEMENT

Some or all data, or models that support the findings of this study are available from the corresponding author upon reasonable request.

NOTATION

The following symbols are used in this paper.

A_{cr_2mm}	= area of the more than 2-mm-wide cracks on the surface of concrete;
$Di(c)$	= damage index of entire structure;
$Di(c)_{elm}$	= damage index of each FE;
J	= creep coefficient of concrete;
L_{cr}	= total length of cracks on the surface of concrete;
L_{cr_2mm}	= total length of the more than 2-mm-wide cracks on the surface of concrete;
l_d	= development length along the reinforcement across a crack;
R_E	= stiffness reduction index of a wall or a column;
S	= bond slip between the concrete and reinforcement;
W_{c0}	= plastic strain energy of concrete;
$\Delta\sigma_{cb}$	= released stress of a crack;
$\Delta\sigma_{ce}$	= concentration stress of a crack; and
τ_b	= bond stress between the concrete and reinforcement.

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Table 1 Analysis cases and results

Analysis attributes and values	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Excitation type	Static pushover		Dynamic: TCU052×100%		Dynamic: TCU052×280%	
Construction and service periods	None	Considered	None	Considered	None	Considered
Maximum total drift (%)	4.00	4.00	1.10	1.01	2.30	2.27
Maximum story drift (%)	10.09	10.40	1.64	1.53	3.70	3.48
(Corresponding story)	(13)	(13)	(12)	(13)	(22)	(21)
Maximum base shear (MN)	380.5	355.1	367.8	303.6	536.3	474.9
Max. crack width at 1.25% story drift (mm)	3.4	3.9	3.1	3.1	4.1	3.6
(Corresponding story)	(3)	(2)	(2)	(1)	(4)	(4)
Max. crack width (mm)	113.9	85.6	13.0	4.7	354.4	102.7
(Corresponding story)	(22)	(13)	(21)	(18)	(1)	(22)
Crack length L_{cr} (km)	28.6	52.1	62.8	91.4	77.7	141.8
0.2 mm-over crack length At the end of service period (km)	--	0.502	--	0.502	--	0.502
2 mm-over crack length $L_{cr\ 2mm}$ (km)	1.226	1.172	0.029	0.022	0.526	0.618
Damage index $Di(c)$	10.57	12.17	0.76	0.65	22.58	8.05
Total CPU time (hour)	63.8	145.7	256.4	379.1	247.1	506.0

Figure 1: DLCM and parallelism: (a) Constitutive models; (b) Bond stress-slip model; (c) redistribution along development length l_d ; (d) released stress $\Delta\sigma_{cb}$; (e) concentration stress $\Delta\sigma_{ce}$; (f) centroid shift of connecting crack; (g) parallelization of conventional FEM; and (h) parallelization of DLCM

Figure 2: Comparison between new and old DLCMs: (a) old DLCM; (b) new DLCM; and (c) nonorthogonal crack coordinates and shear transfer model for new DLCM

Figure 3: Analyzed building: (a) meshing and structural configurations; (b) relationships between shrinkage strains, development ratios, creep coefficients and material age; (c) bond stress-slip curve under reversed cyclic condition; (d) chronological change of bond stress-slip curve; (e) north-east-38-degree component of TCU052 velocity record (for Cases 5 to 6); and (f) TCU052 velocity spectrum

Figure. 4: Analysis results: (a) relationships between base shear, total length of cracks on member surfaces L_{cr} , total length of >2 mm-wide cracks L_{cr_2mm} , damage index $Di(c)$, and total drift of Cases 1 and 2; (b) time histories of total drift, L_{cr} , L_{cr_2mm} , and $Di(c)$ of Cases 3 and 4; (c) time histories of total drift, L_{cr} , L_{cr_2mm} , and $Di(c)$ of Cases 5 and 6; (d) shear, axial force, axial strain, and stiffness reduction index of Wall EF3 of Case 6; and (e) moment, axial force, axial strain, and stiffness reduction index of Column H3 of Case 6

Figure 5: Resume of Case 6: (TCU052×280%): (a) Moment, axial forces and vertical/horizontal strains of Column H3 and Wall EF3 at seventh material day; (b) crack pattern under construction period (25th week); (c) crack patterns at maximum seismic drift (6.75 s); (d) maximum crack width distributions along elevation; and (e) time histories of maximum crack widths of 1st, 25th and 50th stories

Figure 6: Crack characteristics and damage index: (a) 2nd story slab cracks of after service period; (b) wall cracks at 1st and 2nd stories after service period; (c) wall cracks at 1st and 2nd stories of Case 4 at final step; (d) wall cracks at 1st and 2nd stories of Case 6 at final step; (e) definition of plastic strain energy of concrete W_{c0} ; (f) definition and sample of element damage index $Di(c)_{elm}$; (g) $Di(c)_{elm}$ of overall walls of Case 4; (h) $Di(c)_{elm}$ of overall walls of Case 6; (i) relationship between weighed average damage index $Di(c)$ and L_{cr} ; (j) relationship between $Di(c)$ and L_{cr_2mm} ; (k) $Di(c) - L_{cr_2mm}$ correlation at zero-displacement instances and (l) $Di(c) - A_{cr}$ correlation in Walls CD3 and EF3 at 1st and 2nd stories

Concrete	Orthotropic model
Compressive curve	Naganuma 2004
Failure criterion	Willam-Warke 1975 Naganuma 2004
Tension-stiffening	Naganuma 2004
Steel hysteresis	Ciampi 1982
Steel bond	fib 2010 Sato 2014

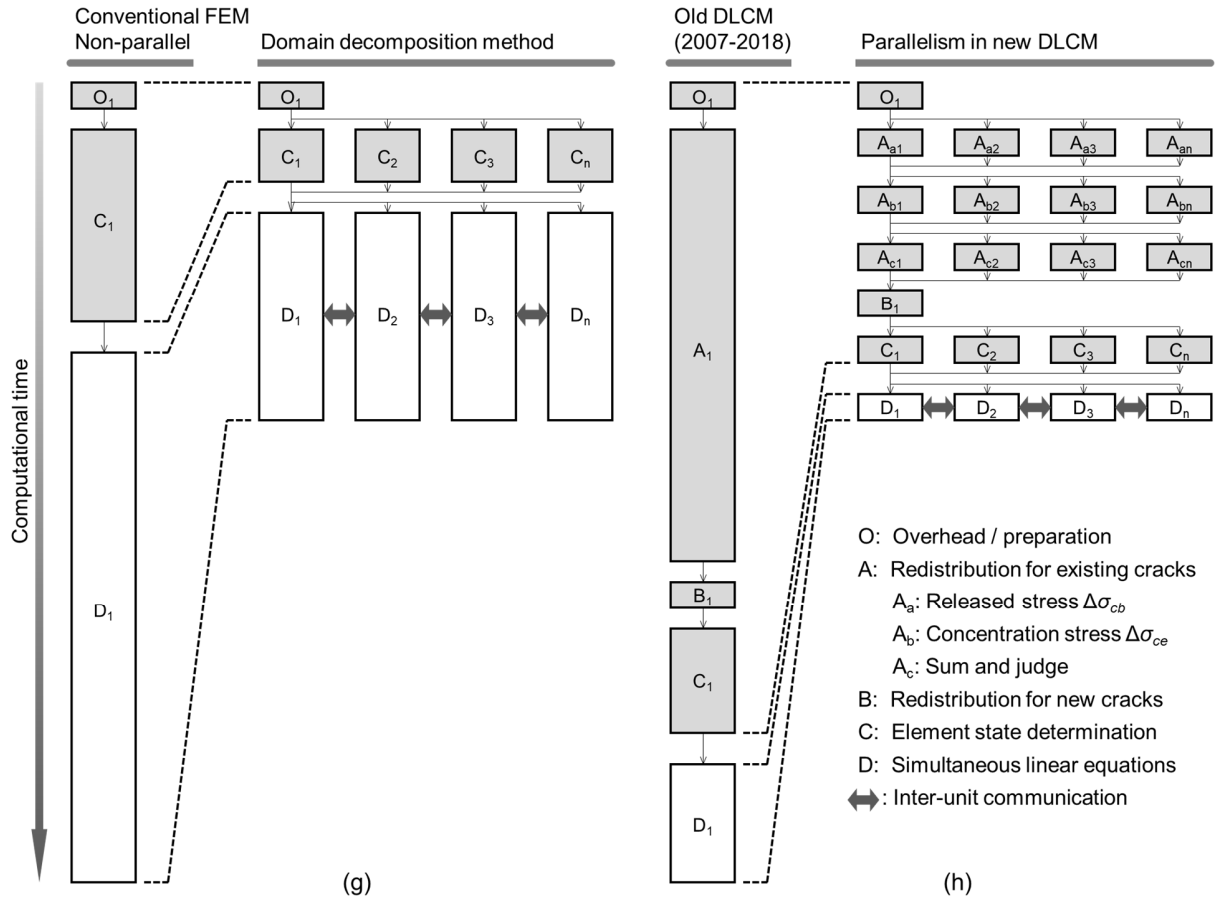
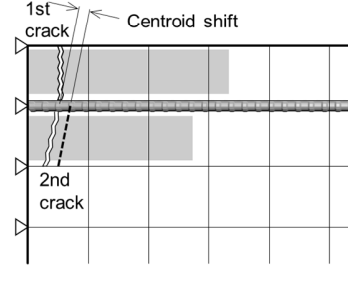
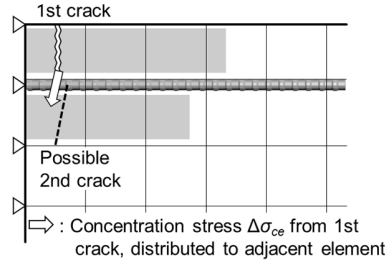
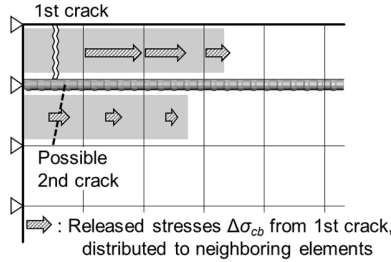
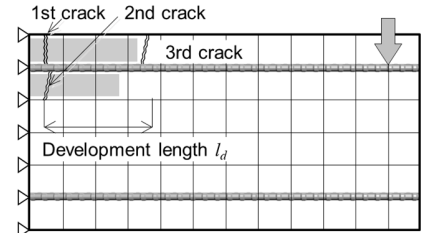
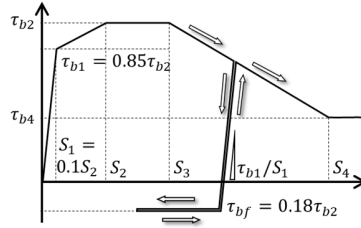
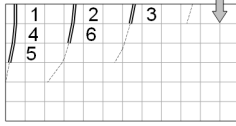


Figure 1: DLCM and parallelism: (a) Constitutive models; (b) Bond stress-slip model; (c) redistribution along development length l_d ; (d) released stress $\Delta\sigma_{cb}$; (e) concentration stress $\Delta\sigma_{ce}$; (f) centroid shift of connecting crack; (g) parallelization of conventional FEM; and (h) parallelization of DLCM

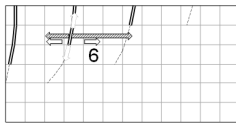
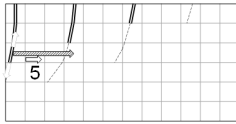
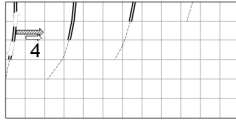
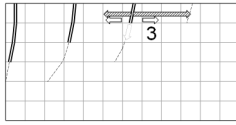
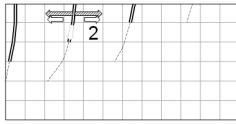
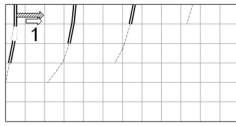
Old DLCM (2007-2020)

Preparation O_1



Non-parallel loop A_1

- Redistribute along reinforcement. Estimate released stress $\Delta\sigma_{cb}$.
- Detect extending crack. Estimate concentration stress $\Delta\sigma_{ce}$.
- Compute $\sigma_{new} = \sigma_{old} + \Delta\sigma_{cb} + \Delta\sigma_{ce}$. If $\sigma_{new} > \sigma_{cr}$, then judge new crack.



Non-parallel loop B_1 : New crack

New crack

Non-parallel Loop C_1 :

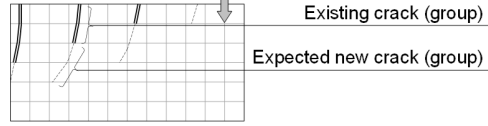
Renew element stiffness

(a)

New DLCM

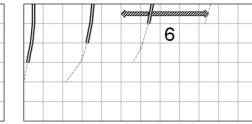
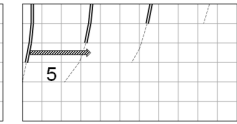
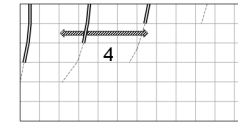
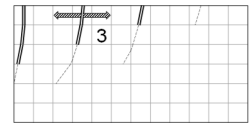
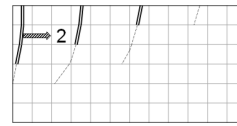
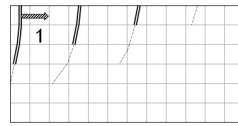
Preparation O_1

Select (i) Elements with existing cracks; (ii) Elements of possible new crack.



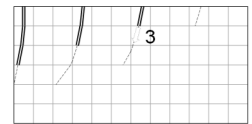
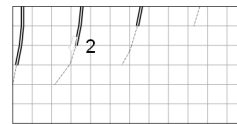
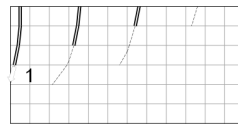
Parallel loops A_{a1} to A_{a6} : Release

- Redistribute along reinforcement; (ii) Estimate minimum released stress $\Delta\sigma_{cbmin}$.



Parallel loops A_{b1} to A_{b3} : Search connectivity

- Detect extending crack series; (ii) Estimate maximum concentration stress $\Delta\sigma_{cemax}$.



Parallel loops A_{c1} to A_{c6} : Sum and judge

- Compute $\sigma_{new} = \sigma_{old} + \Delta\sigma_{cbmin} + \Delta\sigma_{cemax}$; (ii) If $\sigma_{new} > \sigma_{cr}$, then judge new crack.

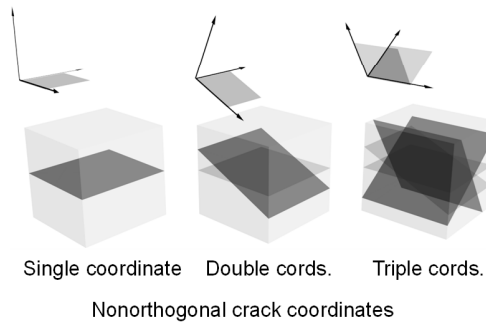
Non-parallel loop B_1 : New crack

- Redistribute stresses for new crack; (iii) Make and renew crack group information.

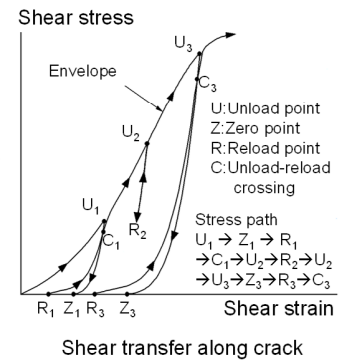
Parallel Loops C_1 to C_{72}

Renew element stiffness.

(b)



Nonorthogonal crack coordinates



Shear transfer along crack

(c)

Figure 2: Comparison between new and old DLCMs: (a) old DLCM; (b) new DLCM; and (c) nonorthogonal crack coordinates and shear transfer model for new DLCM

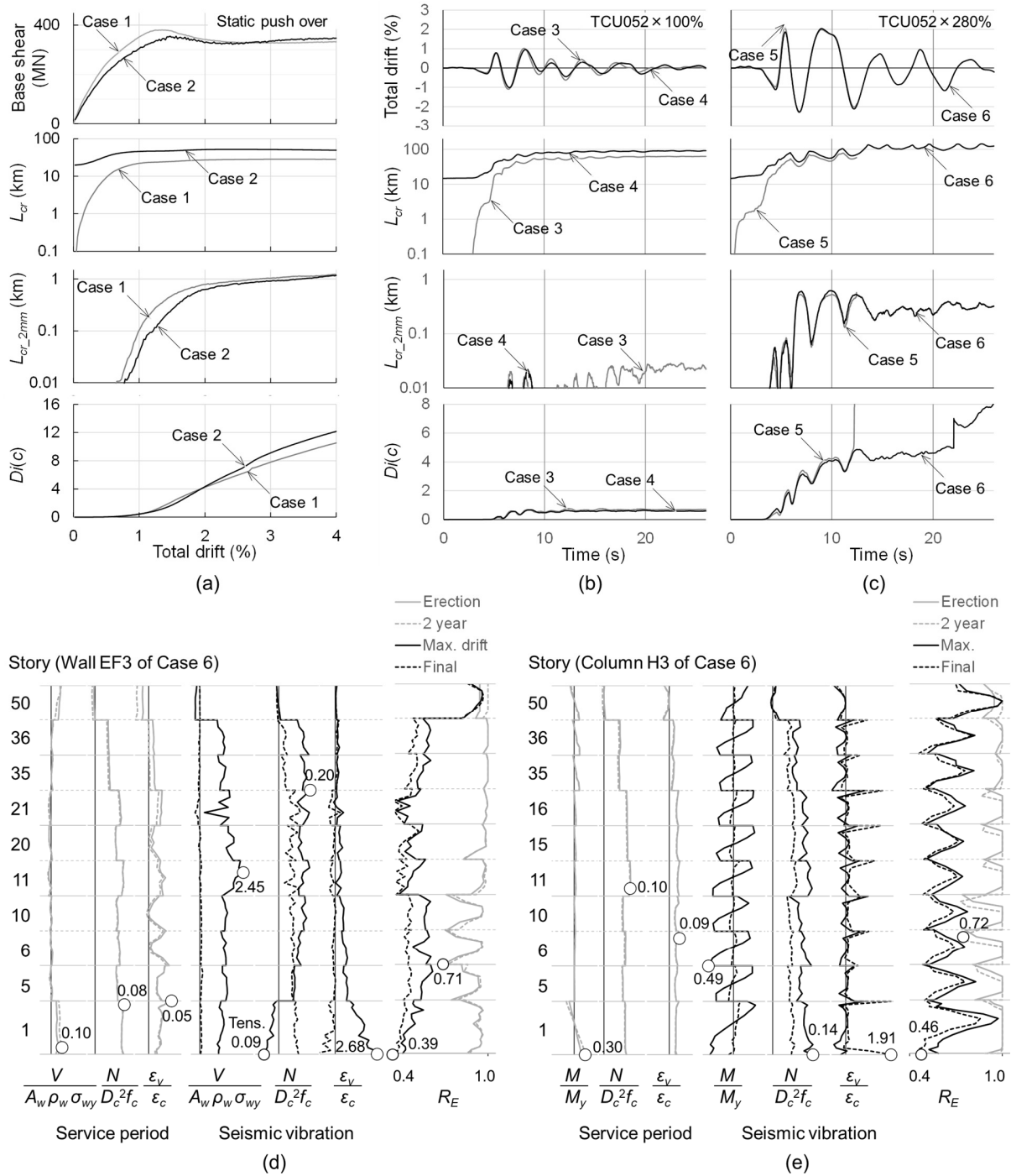


Figure 4: Analysis results: (a) relationships between base shear, total length of cracks on member surfaces L_{cr} , total length of >2 mm-wide cracks L_{cr_2mm} , damage index $Di(c)$, and total drift of Cases 1 and 2; (b) time histories of total drift, L_{cr} , L_{cr_2mm} , and $Di(c)$ of Cases 3 and 4; (c) time histories of total drift, L_{cr} , L_{cr_2mm} , and $Di(c)$ of Cases 5 and 6; (d) shear, axial force, axial strain, and stiffness reduction index of Wall EF3 of Case 6; and (e) moment, axial force, axial strain, and stiffness reduction index of Column H3 of Case 6

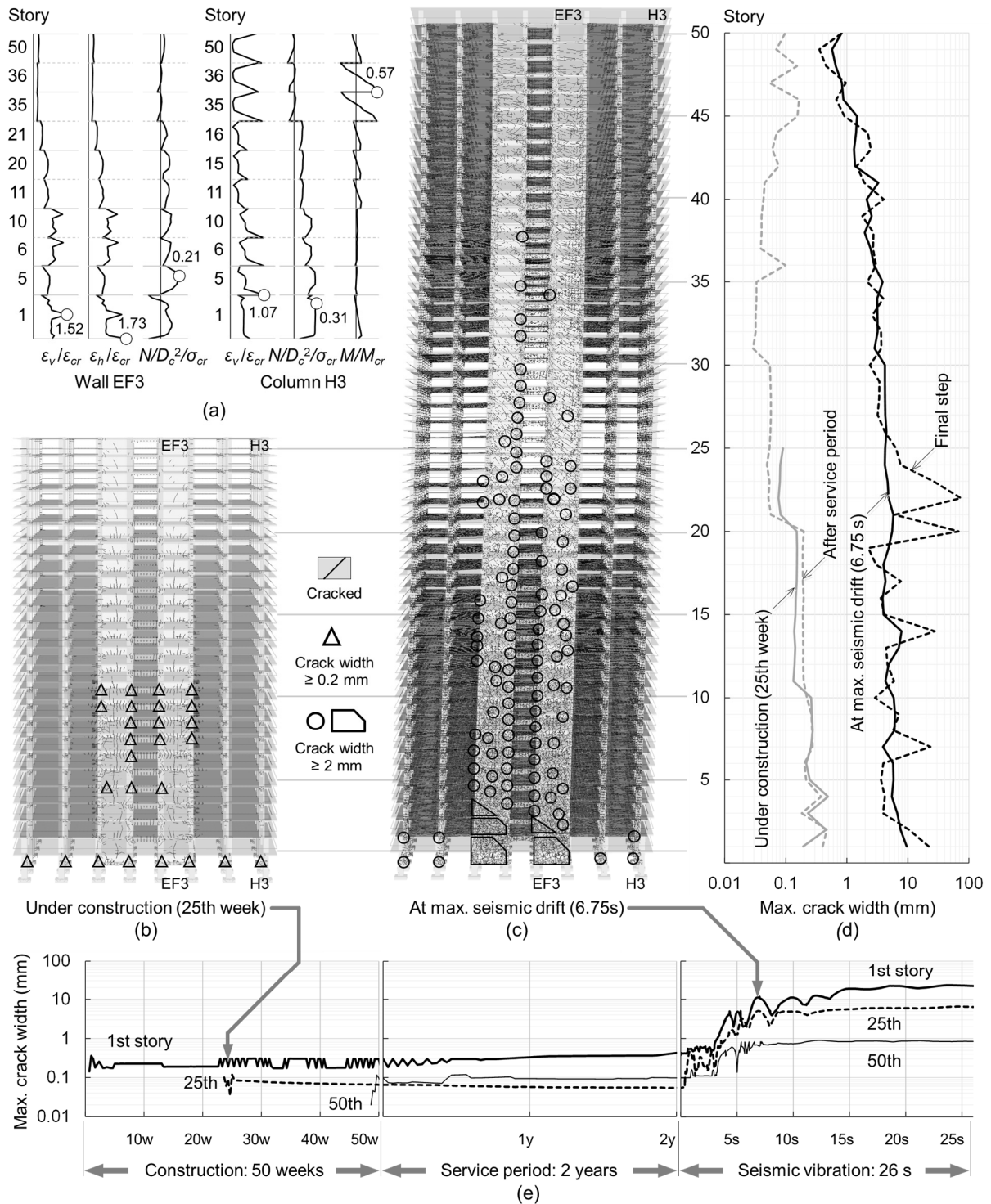


Figure 5: Resume of Case 6: (TCU052×280%): (a) Moment, axial forces and vertical/horizontal strains of Column H3 and Wall EF3 at seventh material day; (b) crack pattern under construction period (25th week); (c) crack patterns at maximum seismic drift (6.75 s); (d) maximum crack width distributions along elevation; and (e) time histories of maximum crack widths of 1st, 25th and 50th stories

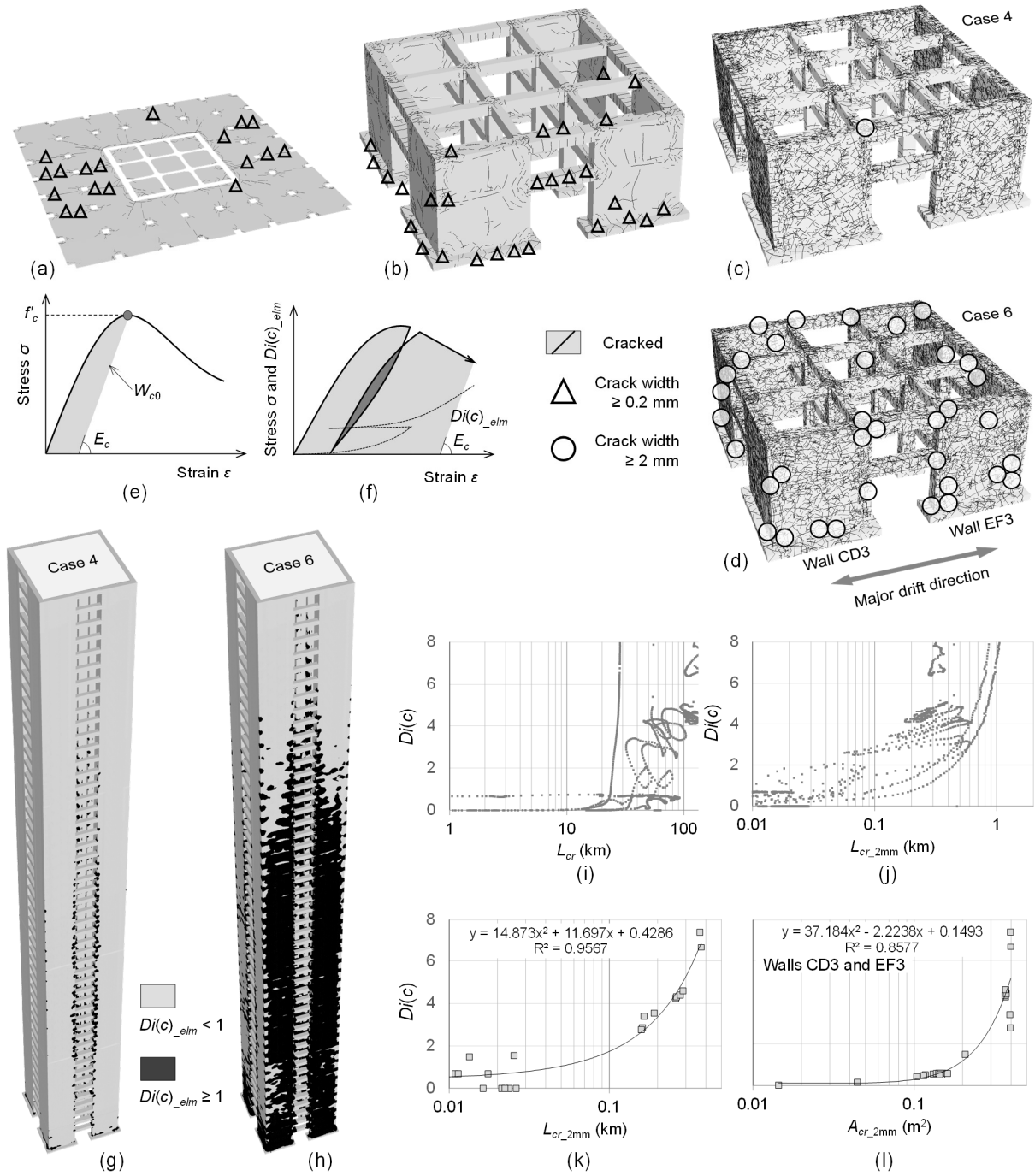


Figure 6: Crack characteristics and damage index: (a) 2nd story slab cracks of after service period; (b) wall cracks at 1st and 2nd stories after service period; (c) wall cracks at 1st and 2nd stories of Case 4 at final step; (d) wall cracks at 1st and 2nd stories of Case 6 at final step; (e) definition of plastic strain energy of concrete W_{c0} ; (f) definition and sample of element damage index $Di(c)_{elm}$; (g) $Di(c)_{elm}$ of overall walls of Case 4; (h) $Di(c)_{elm}$ of overall walls of Case 6; (i) relationship between weighed average damage index $Di(c)$ and L_{cr} ; (j) relationship between $Di(c)$ and L_{cr_2mm} ; (k) $Di(c)$ - L_{cr_2mm} correlation at zero-displacement instances and (l) $Di(c)$ - A_{cr_2mm} correlation in Walls CD3 and EF3 at 1st and 2nd stories