CHAPTER 6

FINITE ELEMENT ANALYSIS OF EXPERIMENT

The FEA (Finite Element Analysis) is applied in a variety of engineering problems because it can handle complex geometry, complex restraints and complex loading with satisfactory accuracy in prediction. In structural engineering, many Finite Element computer programs have been developed and widely used to check the behavior of practical structures, particularly with regard to the evaluation of critical regions with special detailing or regions of stress concentration. Many simulations have shown that the finite element method has the potential to predict the behaviors of a wide range of reinforced concrete walls for which the behaviors have also been investigated through experiment. Satisfactory agreement has been achieved between the behaviors predicted by the analytical method and the experiment, including load-deformation, cracking patterns, load-carrying capacity, and failure modes. Due to the complexity of engineering problems, not all parameters can be investigated in one experiment simultaneously. In addition, high cost of experiments including labor work and money limit the number of tests that can be conducted. It is not economical to inspect all parameters through experiment. Through parametric studies, the FEA can be used to effectively extend the results of omitted experiments and establish a sound theoretical basis for the development of design procedures, and decrease the amount of effort to conduct experiment.

Analytical models for RC structures have generally been based on replacing the reinforced concrete composite continuum by an assembly of finite elements representing the concrete and the steel reinforcement. These models should be able to reflect the behavior of concrete in tension and compression, the response of the reinforcing bars and their interaction with the concrete. Hence, a realistic
stress-strain relationship for both concrete and steel bars and a failure theory are required to obtain basic information through an analytical model of the structure.

The implementation of nonlinear constitutive relations in finite element analysis codes is generally undertaken in one of two ways. In the first case the material behavior is programmed independently of the elements. Using this approach the choice of elements for a particular structural system is not limited and best practice modeling techniques can be used in identifying an appropriate element type to which any of the nonlinear material properties are assigned. This is the most adaptable approach and does not limit the analyst to specific element types in configuring the problem of interest. A second approach is to provide specific nonlinear material capabilities only for dedicated element types. In order to achieve the analytical objective of this study, DIANA 8.1, a nonlinear finite element computer program, was used to simulate the response of the tested specimens. DIANA provides dedicated and two-dimensional eight-node plane stress element and three-dimensional twenty-node solid element, to model the nonlinear response of brittle materials based on a constitutive model for the behavior of concrete.

6.1 Introduction of DIANA

To analyze RC members—beams, shear walls and panels in which the predominant stress state is of plane stress—subject to cyclic loads, it is necessary to describe accurately the cyclic behavior of concrete which is anisotropically damaged by multiple tensile cracks. For this purpose, many constitutive models have been proposed for the nonlinear finite-element analysis of reinforced concrete structures under plane stress conditions. These can be classified into orthotropic models, nonlinear elastic models, plasticity models, endochronic models, fracture mechanics models, and micromodels such as the microplane and nonlocal
continuum model. In DIANA, an isotropic model is used to simulate behavior of concrete.

Under loading, micro- and macro-cracks occur in concrete. The stress-strain curve of concrete is influenced by the development of these cracks. Discrete and smeared crack approaches are the main two methods to simulate the cracking behavior of concrete.

In the discrete crack approach that was first introduced to concrete structures by Saouma and Ingraffea (1981), the discontinuities of the displacement field resulting from the failure process are introduced directly into the numerical model. The discrete approach is based on the principles of fracture mechanics or the fictitious crack concepts. This method is theoretically more suitable to capture failure localization.

Smeared crack models, first introduced by Rashid (1968), are based on the development of appropriate continuum material models, in which cracks are smeared over a distinct area, typically over a finite element or an area corresponding to an integration point of the element. It builds up on an equivalent continuum concept of elastic degradation and/or softening plasticity within the fixed mesh approach.

Among concrete constitutive relations orthotropic models strike a balance between accuracy and economy. These models are generally based on the concept of “equivalent uniaxial strain” by Darwin and Pecknold (1977). The models proposed to date differ in the description of the biaxial failure envelope, the uniaxial equivalent stress-strain relation, poisson ratio, and the tension-compression behavior. From an extensive experimental study by Vecchio and Collins (1981), Vecchio
(1992) developed an orthotropic concrete model with compressive strength degradation as a function of tensile strain after cracking. In another model by Balakrishnan and Murray (1998) the degradation in compressive strength is a function of tensile stress at cracking.

In this analysis, DIANA uses the constitutive model based on total strain which is developed along the lines of the Modified Compression Field Theory (MCFT), originally proposed by Vecchio and Collins. The constitutive models based on total strain describe the tensile and compressive behavior of a material with one stress-strain relationship. These models can neither be combined with other constitutive models, nor with ambient influence. This makes the models very well suited for serviceability limit state and ultimate limit state analyses which are predominantly governed by cracking or crushing of the material.

The basic input for the crack models based on total strain consists of two parts:
(1) The basic properties like Young’s modulus, Poisson’s ratio, tensile and compressive strength,
(2) The definition of the behavior in tension, shear and compression.

DIANA supplies a set of pre-defined tension softening functions. Some of these softening curves are based on fracture energy by the definition of the crack band width of the element, for which DIANA assumes a value related to the area or the volume of the element.

To model the lateral confinement effect, the parameter of the compressive stress-strain function are determined with a failure function which gives the compressive stress which causes failure as a function of the confining stresses in the lateral direction.
The determination of crack initiation is another important feature of this model. In DIANA, the initiation of a crack is governed by tension cut-off criterion and a threshold angle between two consecutive cracks. For successive initiation of the cracks, DIANA applies two criterions which must be satisfied simultaneously:

1. The principal tensile stress violates the maximum stress condition,
2. The angle between the existing crack and the principal tensile stress exceeds the value of a threshold angle.

A total strain-based constitutive model describes the stress as a function of the strain. This concept is known as hypo-elasticity when the loading and unloading behavior is along the same stress-strain path. But in the implementation in DIANA, the behavior in loading and unloading is modeled differently with secant unloading. Basically, the nonlinear response is caused by two major material effects, cracking of the concrete and plasticity of the reinforcement and of the compression concrete.

### 6.2 Model of Reinforced Concrete Walls for Finite Element Analysis

Six specimens tested in this study were simulated using DIANA 8.1. To investigate the effects of different configuration of steel shapes on performance of walls with different steel sections, two-dimensional models were built for modeling behavior of specimen W1, W2, W3 and W4 and three-dimensional models were built for specimen W1, W4 and W5 and W6.

#### 6.2.1 Material Model
A. Concrete

- Compression Behavior

Concrete subjected to compressive stresses shows a pressure-dependent behavior, that is, the strength and ductility increase with increasing isotropic stress. Due to the lateral confinement, the compressive stress-strain relationship is modified to incorporate the effects of the increased isotropic stress. Furthermore, it is assumed that the compressive behavior is influenced by lateral cracking.

- Tensile Behavior

The tensile behavior of reinforced concrete can be modeled using different approaches. For the total strain crack model, four softening functions based on fracture energy are implemented, a linear softening curve, an exponential softening curve, the nonlinear softening curve according to Reinhardt et al., and the nonlinear softening curve according to Hordijk, all related to a crack band width as is usual in smeared crack models.

Hordijk, Cornelissen and Reinhart proposed an expression for the softening behavior of concrete, as shown in Fig.6.2.1-1, which also results in a crack stress equal to zero at a crack strain of \( \varepsilon_{\text{cr,ult}} \). The function is defined by

\[
\frac{\sigma_{\text{cr}}(\varepsilon_{\text{cr}})}{f_t} = \begin{cases} 
1 + \left( \frac{c_1 \varepsilon_{\text{cr}}}{\varepsilon_{\text{cr,ult}}^t} \right)^2 \exp \left(-c_2 \frac{\varepsilon_{\text{cr}}}{\varepsilon_{\text{cr,ult}}^t} \right) & 0 < \varepsilon_{\text{cr}} < \varepsilon_{\text{cr,ult}}^t \\
0 & \varepsilon_{\text{cr,ult}}^t < \varepsilon_{\text{cr}} < \infty 
\end{cases} \tag{6-1}
\]

With parameters \( c_1 = 3 \) and \( c_2 = 6.93 \)

\[
\varepsilon_{\text{cr,ult}} = 5.136 \frac{G_f'}{h f_t} \tag{6-2}
\]
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Fig. 6.2.1-1 Nonlinear Tension Softening (Hordijk et al.)

- Shear Behavior
  The modeling of the shear behavior was only necessary in the fixed crack concept where the shear stiffness was usually reduced after cracking. In DIANA, only a constant shear stiffness reduction is modeled,

  \[ G' = \beta G \]  

  with \( \beta \) the shear retention factor, \( 0 \leq \beta \leq 1 \).

  For the rotating crack concept the shear retention factor can be assumed equal to one.

  Shear modulus \( G \):

  \[ G = \frac{E}{2(1+v)} \]  

  \( E \): modulus of elasticity
  \( v \): poisson's ratio

B. Steel

- Yield Criteria
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The yield condition of Von Mises is a smooth approximation of the Tresca yield condition: a circular cylinder in the principal stress space. The yield function of Von Mises is:

\[ f(\sigma, \kappa) = \sqrt{3J_2} - \sigma(\kappa) \quad (6-4) \]

Where \( \sigma(\kappa) \) is the uniaxial yield strength as a function of the internal state variable \( \kappa \).

- **Hardening**

The relation between the internal state variables \( \kappa \) and the plastic process is given by the hardening hypothesis: strain hardening and work hardening.

In the present study, the strain hardening is selected:

\[ \dot{\kappa} = \frac{1}{\lambda} \left( \dot{\varepsilon}_1^p \dot{\varepsilon}_1^p + \dot{\varepsilon}_2^p \dot{\varepsilon}_2^p + \dot{\varepsilon}_3^p \dot{\varepsilon}_3^p \right) \quad (6-5) \]

Which can be reduced to

\[ \dot{\kappa} = \dot{\lambda} \quad (6-6) \]

For the Von Mises yield condition, the translation of uniaxial experimental data to the equivalent stress-internal state variable, the \( \bar{\sigma} - \kappa \) relation, is independent of the hardening hypothesis.

As shown in Fig.6.2.1-2, the plastic strain \( \varepsilon_i^p \) is assumed to be given by \( \varepsilon_i - \varepsilon_i^e \). The uniaxial stress-plastic strain diagram is shown in Fig.6.2.1-3. The uniaxial plastic strain rate is given by

\[ \dot{\varepsilon}_i^p = \dot{\lambda} \frac{\sigma_i}{\bar{\sigma}} \quad (6-7) \]

The relation between the uniaxial stress and the equivalent stress is simply

\[ \bar{\sigma} = \sigma_1 \quad (6-8) \]

Therefore followings can be derived:

\[ \dot{\kappa} = \dot{\varepsilon}_i^p \]
Fig. 6.2.1-2 (a) Uniaxial Stress-Strain Curve,
(b) Uniaxial Stress-Plastic Strain Curve
C. Interface Element

In general, performance of the interface between two parts of a structure is governed by frictional behavior. This behavior can be modeled with the Coulomb friction model, which has close resemblance to the Mohr-Coulomb plasticity model for continuum elements in this study. Behavior of the interface between the upper and lower wall panels can be simulated by interface elements with Coulomb frictional material properties. Therefore, the characteristics of Mohr-Coulomb plasticity for concrete need to be studied.

(1) Mohr-Coulomb Yield Criterion (Concrete)

As material is in multi-axial states of stress, failure occurs at the initiation of inelastic material behavior through either yielding or fracture. The study of the behavior of materials that yield is known as plasticity theory. In general, a complete plasticity theory has three components: a yield criterion that defines the initiation of yielding in a material, a flow rule that relates the plastic strain increments to the stress increments after initiation of yielding, and a hardening rule that predicts changes in the yield surface.
The yield behavior of concrete, as a cohesive material, has been observed to be dependent on hydrostatic stress. Specially, an increase in hydrostatic compressive stress produces an increased ability of some materials to resist yielding. Also, it exhibits different yield stresses in tension and compression.

The yield condition of Mohr–Coulomb is an extension of the Tresca yield condition to a pressure dependent behavior accounting for the influence of hydrostatic stress as shown in Fig.6.2.1-4 and Fig.6.2.1-5. The formulation of the yield function can be expressed in the principal stress space \((\sigma_1 > \sigma_2 > \sigma_3)\) as

\[
f(\sigma, k) = \frac{1}{2} (\sigma_1 - \sigma_3) + \frac{1}{2} (\sigma_1 + \sigma_3) \sin \phi(k) - c(k) \cos \phi(k)
\]

with \(c(k)\) the cohesion as a function of the internal state variable \(k\), and \(\phi\) the angle of internal friction which is also a function of the internal state variable. The initial angle of internal friction is given by \(\phi_0\). The flow rule is given by a general non-associated flow rule \(g \neq f\), but with the plastic potential given by

\[
g(\sigma, k) = \frac{1}{2} (\sigma_1 - \sigma_3) + \frac{1}{2} (\sigma_1 + \sigma_3) \sin \varphi(k)
\]

which results in the plastic strain rate vector:

\[
\dot{\epsilon}^p = \dot{\Lambda} \left\{ \begin{array}{c}
\frac{1}{2} (1 + \sin \varphi) \\
0 \\
\frac{1}{2} (1 - \sin \varphi)
\end{array} \right\}
\]

(6-11)
Where $\phi$ represents the angle of internal friction of the sample; $C$ represents the cohesion (or shear strength intercept); and $T$ is the tensile strength. A tensile limit, $T_0$, is used when the tensile strength of the material is lower than the strength defined by the failure criterion.

- Hardening

The relation between the internal state variable $k$ and the plastic process is given by the hardening hypothesis. For the Mohr–Coulomb yield condition only the strain hardening hypothesis is taken into account.
In the case of strain hardening the relation is given in the principal space by
\[
\dot{\epsilon} = \sqrt{\frac{2}{3}} \left( \dot{\epsilon}_{p,1}^p \dot{\epsilon}_{1,1}^p + \dot{\epsilon}_{p,2}^p \dot{\epsilon}_{2,2}^p + \dot{\epsilon}_{p,3}^p \dot{\epsilon}_{3,3}^p \right)
\] (6-12)
which can be reduced to
\[
\dot{\epsilon} = \dot{\lambda} \sqrt{\frac{1}{3}} \left( 1 + \sin^2 \phi \right)
\] (6-13)

- Relation \(\bar{c} - k\)

The translation of uniaxial experimental data to the equivalent cohesion–internal state variable, the \(\bar{c} - k\) relation, depends on the hardening hypothesis. In the following example, a derivation is given for a cohesion hardening material with constant friction and dilatation angle, \(\phi(k) = \phi_0\) and \(\phi(k) = \varphi_k\), and a strain hardening hypothesis.

Consider the uniaxial stress–strain diagram \(\sigma_3 - \epsilon_3\) of Fig.6.2.1-6a. The plastic strain \(\epsilon_p^p\) is assumed to be given by \(\epsilon_3 - \epsilon_3^f\). Fig.6.2.1-6b shows the uniaxial stress–plastic strain diagram. For uniaxial stressing, \((\sigma_1, \sigma_2, \sigma_3) = (0,0,\sigma_3)\) plastic flow occurs at a vertex of the yield surface. Symmetry conditions dictate that the two possible yield directions contribute equally to the plastic strain rate vector
\[
\dot{\epsilon}^p = \begin{bmatrix} \dot{\epsilon}_{1,1}^p \\ \dot{\epsilon}_{2,2}^p \\ \dot{\epsilon}_{3,3}^p \end{bmatrix} = \dot{\lambda} \begin{bmatrix} \frac{1}{4} (1 + \sin \varphi_0) \\ -\frac{1}{2} (1 - \sin \varphi_0) \end{bmatrix}
\] (6-14)

With the relation derived previously, the relation between the uniaxial plastic strain and the internal state variable for a strain hardening hypothesis is:
\[
\dot{k} = -2 \sqrt{\frac{1}{3} \left( 1 + \sin^2 \varphi_0 \right)} \frac{1}{1 - \sin \varphi_0} \dot{\epsilon}_3^p
\] (6-15)
The relation between the uniaxial stress $\sigma_3 = -f_c$ and the equivalent cohesion $\overline{c}$ is given by

$$\overline{c} = f_c \frac{1 - \sin \phi'_0}{2 \cos \phi'_0}$$  \hspace{1cm} (6-16)

Fig.6.2.1-6 illustrates the procedure for $\phi'_0 = \phi'_0 = 30^\circ$

(2) Coulomb Friction Model

In interface elements with linear material models, the slip along the I-Beam which occurs when inner stresses reached yielding criterion can not be simulated with a complete accuracy,. The gap that occurs when the tension stress in the normal direction exceeds $f_t$, also can not be predicted by the linear frictional material model. Aiming at simulating the slip and gap, which has been observed in experiment, a nonlinear friction material model was introduced into the study.
Development of design methods for interfaces between granular soils and civil construction materials depends on understanding how particular strength and volume change behavior is influenced by the geometry of rigid interfaces. As shown in Fig.6.2.1-7, a conceptual model for interlocking friction consists of two serrated surfaces pressed together by a normal load, $Q$. Resistance, $P$, to relative motion in plane shear is given by:

$$P = Q \tan(\phi_s + \phi)$$  \hspace{1cm} (6-17)

where $\phi_s$ is the friction angle between the two materials. An additional force component must be applied to permit expansion (dilation) at angle $\phi$ against the normal load, $Q$. A model for plane shear of a granular assemblage is obtained by replacing the upper serrated surface with particles as shown in Fig.6.2.1-8. The peak shear resistance of the assemblage is given by:

$$\tau = \sigma \tan(\phi_s + \phi)$$  \hspace{1cm} (6-18)

where $\tau$ is the peak state shear stress, $\sigma$ the normal stress, $\phi_s$ the steady state friction angle of the granular material, and $\phi$ the dilation angle. Dilation angle is determined from the slope of the volume change versus horizontal displacement curve. $\tan \phi = \frac{dy}{dx}$ Positive dilation values indicate volume expansion whereas negative values indicate volume contraction. The dilation angle is greatest at peak states and diminishes to zero as steady-state is reached.
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For a line interface element, we have the following traction vector and relative displacement vector:

\[
t = \begin{bmatrix} t_n \\ t_s \end{bmatrix}, \quad \Delta u = \begin{bmatrix} \Delta u_n \\ \Delta u_s \end{bmatrix}
\] (6-20)

where \( t_n \) is the normal traction across the interface and \( t_s \) is the shear traction. In the reference configuration, the relative displacement between the two faces is equal to zero. The opening component of the relative displacement is denoted by \( \Delta u_n \) and the sliding component by \( \Delta u_s \).

It is assumed that irreversible displacements occur when the two faces of the interface element are in contact. Consequently, the additive decomposition of the relative displacements into an elastic and plastic part is assumed, \( \Delta u = \Delta u^e + \Delta u^p \). A simple, uncoupled relation between the elastic components of the relative displacements and the tractions is

\[
t = D \Delta u^e
\]

where \( D = \begin{bmatrix} d_{nn} & 0 \\ 0 & d_{ss} \end{bmatrix} \)

The Coulomb friction material model can be formulated a yield surface

\[
f = |t_s| + t_n \tan \phi - \bar{c}(k)
\] (6-21)

with a friction angle \( \phi \) and a cohesion \( \bar{c}(k) \) as a function of the hardening parameter \( k \). If the traction vector lies within the yield surface, i.e., if \( f < 0 \), the inelastic relative velocity vector, \( \Delta \dot{u}^p \), is equal to zero. If the traction vector lies on the yield surface, i.e., if \( f = 0 \), the inelastic relative velocity vector may be nonzero, and is determined by a nonassociated flow rule as shown in Fig.6.2.1-9,

\[
g = |t_s| + t_n \tan \phi - \bar{c}
\] (6-22)
with a dilatation angle $\varphi$. Physical considerations require that $0 \leq \varphi \leq \phi$. The inelastic part of the relative velocities must then satisfy the relation

$$\Delta u_n^p = |\Delta u_j^p| \tan \varphi$$

(6-23)

Fig. 6.2.1-9 Coulomb friction model

(3) Friction Angle and Dilation Angle in Past Research

Angle of internal friction can be calculated only if a series of uniaxial and tri-axial tests are performed on samples of the material in question. Because the material properties of concrete have been documented by a number of authors, these time consuming and expensive tests were not conducted for the concrete block. Rather, a value of $26.5^\circ$ was chosen. Giaccio and Zerbino (1998) found this angle of internal friction applied to conventional concrete is independent of the aggregate used in production of the concrete.

Dilatancy is a measure of the change in volume that occurs when shear stress is applied to a material. This change is characterized by a dilation angle, $\varphi$, which is the ratio of volume change to shear strain. Vermeer and de Borst (1984) found $12^\circ$ to be a typical dilation angle for concrete and this value was used in the single block model.
The coefficient of friction, cohesion, and stiffness of the interfaces between the steel platens and concrete block have an influence on the behavior of the model and have to be defined. Some tests conducted with concrete masonry unit blocks similar to those used in the tests have shown the coefficient of friction to be 0.3 between steel and concrete and 0.5 between two concrete blocks (Gearhart 2003). This corresponds to a friction angle of $17^\circ$ for the black-platen interfaces and $26.5^\circ$ for the concrete block interfaces.

### 6.2.2 Meshing

The appropriate element layout and size (meshing) is an important aspect in finite element simulation to capture the behaviors of interest. If the mesh is too fine, one might get singularities in the solution, stress concentrations which should not appear, or the solver will take an extraordinary long time to run.

DIANA provides several tensile models for concrete which are based on fracture energy. Fracture energy was released in any element if the tensile strength is violated and the deformations localize in the element. With this approach, the results which are obtained from the analysis are objective with regard to mesh refinement. Sometimes it is possible that the elements of the discretization are so large that the equivalent length of an element results in a snap-back in the constitutive model and the concept of objective fracture energy which has been assumed is no longer satisfied.

Aiming at good convergence, the tension softening curve based on energy is utilized, that is, tensile strength with regard to tensile strain is a function of fracture energy and crack band width.
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\[ f_t = F \left( \frac{G_f'}{h} \right) \]  \hspace{1cm} (6-24)

Where \( f_t \) = tensile strength of concrete.

\( G_f' \) = fracture energy.

\( h \) = the crack bandwidth, default value for linear two-dimensional elements is \( h = \sqrt{2A} \), \( A \) is the total area of the element.

Attention needs to be paid to the following terms, when the walls are discretized:

1) Concrete and reinforcement use different elements, using plane stress elements for concrete walls and truss elements for reinforcement, respectively.

2) The meshing of the concrete wall is concerned with the fracture energy mentioned above. It is recommended that the side length of a RC plate should be with a range of limit dimensions. Sometimes zigzag exists in the load-displacement curve, this can be attributed to too large of a dimension of the concrete element which causes the analysis to be unstable.

3) The reinforcement can be modeled using truss elements, and nodes of these truss elements are merged into the nodes of the concrete elements. So, these reinforcement elements must be placed on the boundary of the concrete elements. This kind of model of interaction between concrete and steel reinforcement is based on assuming that the steel and concrete has perfect bond. The reinforcement can also be modeled using truss elements and the interaction between concrete and reinforcement can be modeled using an interface element. Embedded reinforcement can also be used to model the steel bar in the reinforced concrete members.

4) In the analysis of this research, to get convenient placement of truss
elements to represent reinforcement, the rectangular plane stress elements were adopted to model concrete wall.

6.3 Simulation of Specimen W1

6.3.1 2-D Model

A. Elements Types

Shear walls are known as plane stress members, a two-dimensional model was built to simulate the behavior of the wall. Load was applied to the wall that corresponded with the values measured during the displacement cycles.

The concrete was simulated by CQ16M which was an eight-node quadrilateral isoparametric plane stress element. It was based on quadratic interpolation and Gauss integration. Horizontal and vertical reinforcement was represented by the L2TRU element which is a two-node directly integrated (1-point) truss element.

Diagonal reinforcement in this specimen was simulated by embedded reinforcement which adds stiffness to the finite element model. The features of embedded reinforcement are:

- Reinforcement embedded in structural elements are called mother elements.
- Reinforcement does not have degrees of freedom of their own.
- By default, reinforcement strains are computed from the displacement field of the mother elements. This implies perfect bond between the reinforcement and the surrounding material.

B. Specimen Mesh

In this model, channels embedded at boundaries of the wall were represented by truss elements with same sectional area. Diagonal reinforcement in the lower
part of wall was simulated by embedded reinforcement with perfect bond. Considering previous conditions on meshing specimen, the specimen was meshed as shown in Fig.6.3.1-1 in which the concrete element was surrounded by steel truss elements as shown in Fig.6.3.1-2. Thicknesses of different components were given to represent the wall, top beam and base beam. The bottom of the foundation beam was constrained along the x, y directions to simulate the fixed boundary conditions as done in the experiment.

![Screen Shot 2021-02-27 at 3.10.22 PM](image.png)

**Fig.6.3.1-1 Mesh of Specimen W1**

**Fig.6.3.1-2 Layout of Reinforcement and Concrete**

### 6.3.2 2-D Analytical Results

As shown in Fig.6.3.2-1, comparison of experimental and analytical results
shows that seismic characteristics of the wall including yield strength and and failure strength, loading and unloading stiffness, ductility and pinching are accurately captured.

It can be seen from Fig.6.3.2-2 that in the early loading stage, when the load was equal to \(-0.25P_t\), there was diagonal principal compressive stress starting from the loading point to the opposite lower corner of the wall, which means that at this stage the diagonal concrete strut shown in Fig.6.3.2-2 was the main load resistant component. When the load increased to \(-P_t\), as given in Fig.6.3.2-3, the compressive concrete band became wider. Two new diagonal compressive stress bands appeared, one of which started from the lower corner of the wall to the opposite mid-height of the wall boundary, the other from the mid-height to the loading point. And in this stage the contribution of diagonal reinforcement to resist lateral load was not evident.

With increasing the displacement level, when the ductility reached 5, the contribution of diagonal reinforcement near the foundation became apparent. These diagonal bars can carry tension or compression. When displacement ductility reached -8, Fig.6.3.2-4 shows the principal compressive stress at this stage, one diagonal concrete strut starting from middle of the foundation block represents the contribution of the diagonal bars to resist shear load. Another feature was the stress in the left lower corner of the wall decrease to lower level. Referring to the strength of the wall at this displacement shown in Fig.6.3.2-1, it can be deduced that crushing of the concrete occurred in this corner. The contribution of diagonal reinforcement to tension can be seen from Fig.6.3.2-5. Four diagonal bars were in tension whereas the other four diagonal bars were in compression, which contributed to the lateral load resistance at the sections crossed by these diagonal bars.
The stress distribution in the horizontal reinforcement is shown in Fig.6.3.2-6. It can be seen from this that the horizontal reinforcement yielded over more than 2/3 of height of the wall. This agreed with the experimental readings of the strain gauges attached to the horizontal reinforcement. The maximum stress in tension was 501Mpa which occurred in horizontal reinforcement. The maximum stress in compression was -506Mpa which appeared in the vertical reinforcement in the compression zone. Deformation of the specimen at a ductility -8 is shown in Fig.6.3.2-7. Evident horizontal slip occurred at the 50mm height element near the wall foundation interface.

Analytical results showed good agreement with the experimental results including strength, loading and unloading stiffness, ductility, pinching, deformations, and internal force paths shown as principal compressive stress in the concrete, and stresses in the horizontal and diagonal bars. This means that the perfect bond between embedded channels and their surrounding concrete was correctly assumed. The analytical results were in accordance with the experimental observation that no evident relative slip occurred between the channels and the surrounding concrete.
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Fig. 6.3.2-2 Principal Compressive Stress at $-0.25P_l$

Fig. 6.3.2-3 Principal Compressive Stress at $-P_l$
Chapter 6 Finite Element Analysis of Experiment

Fig. 6.3.2-4 Principal Compressive Stress at Ductility -8

Model: SFV/EWW/1-8
LC1: load case 1
Step: 3110 LOAD: 33.4
Gauss RE: SXX GSXX
Max/Min on model set:
Max = 352  Min = 350
Results shown:
Mapped to nodes

Fig. 6.3.2-5 Stress Distribution in Diagonal Reinforcement at Ductility -8

Fig. 6.3.2-6 Stress Distribution in Vertical and Horizontal Bars at Ductility -8

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6.3.3 3-D Model and Results

A. Meshing of Specimen W1

To compare configuration effects of the channels on performance of the specimens, a three-dimensional model was also built as shown in Fig.6.3.3-1. Solid element CHX60 with 20-node was used to simulate concrete, and L2TRU with 2-node was used to represent the horizontal and vertical reinforcement, shell element CQ40 with 8-node was used to simulate the channel embedded in the boundaries of the wall; embedded reinforcement was used to simulated diagonal reinforcement near the base. In this model, steel channels, which were simulated by curved shell elements with equal sectional area as shown in Fig.6.3.3-2, were assumed to have perfect bond with the surrounding concrete with regard to no apparent cracks along the internal boundaries of the channel throughout the experimental test. The material properties used in this 3-D model was same as those in the 2-D model.
B. Analytical Results

Comparison of analytical and experimental results is shown in Fig.6.3.3-3. It can be seen from this figure that the analytical and experimental results exhibit a good agreement in term of strength, loading and unloading stiffness and ductility. The 3-D model exhibited nearly the same prediction of the performance of specimen W1 as was done by 2-D model, including yielding strength and ultimate strength, ductility, and stiffness, and pinching. It shows that in simulating the behavior of shear walls with channels acting as flexural reinforcement, plane stress members, 2-D models and 3-D models demonstrate similar performance with a good agreement.

Fig.6.3.3-1 Three Dimensional Model
6.3.4 Summary

Behavior of specimen W1 under cyclic load was simulated by two-dimensional and three-dimensional models. From comparing hysteresis loops obtained from experiment and FEA, it can be seen that the two models can capture the behaviors of this specimen with a good accuracy. It also shows that in simulating 2-D behavior of plane stress reinforced concrete structural members, two-dimensional
models can be used to predict the behavior of these members with satisfactory agreement with experimental results. The channels acting as flexural reinforcement embedded in boundaries of the wall can be represented by truss elements with the same sectional areas. At different loading stages, the internal load paths exhibit good agreement with truss models based on experimental observations and records of strain gauges and LVDTs.

6.4 Simulation of Specimen W2

6.4.1 2-D Model

A. Meshing of Specimen W2

In simulating the behavior of specimen W2 under cyclic loads, a two-dimensional model was built as shown in Fig. 6.4-2, in this model, line interface element CL24I with 3+3 node shown in Fig. 6.4-1, was applied to simulate the behavior of the gap between the upper and lower wall panels. Diagonal reinforcement in the horizontal connection was represented by embedded reinforcement elements which passed through the horizontal connection. Mesh of specimen W2 is shown in Fig.6.4.1-2.

![Fig.6.4-1 Line Interface Element CL24I](image-url)
B. Properties of Material

A horizontal crack was observed in the connection after several loading cycles. In order to accurately simulate the behavior of the horizontal connection, material of interface element should have such properties to capture features of the gap. In the cyclic loading course, the horizontal crack along the horizontal connection experienced opening and closing. During the loading course, the crack opened due to tension, and part of the crack closed due to compression. In this model, aimed at simulating the actual manner of the crack and considering the mechanism of the interface actions, material model friction was utilized. Gap opening and closing was considered in this simulation by a limit tension strength value. If the gap occurs, the friction in part of the interface loses its ability to resist shear force along the horizontal crack.

Another point under consideration was the performance of the mechanical components in the horizontal connection. As in steel structures, the tolerance of the bolt holes and bolts is taken into account through decreasing stiffness of the diagonal reinforcement in the horizontal connection as shown in Fig.6.4-3, in which one millimeter tolerance was evenly distributed into 1000mm. Therefore the yield
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strain was 0.001 later than test results. Similar treatment has done for flexural connectors.

The two material models introduced here were used in the simulation of specimens W3, W5 and W6.

![Stress-Strain Curve for Diagonal Reinforcement in Horizontal Connection](image)

**Fig.6.4-3 Stress-Strain Curve for Diagonal Reinforcement in Horizontal Connection**

### 6.4.1 Analytical Results

The comparison between the analytical and experimental results is shown in Fig.6.4.1-1. It can be seen from this figure the ultimate strength, unloading stiffness, ductility and strength at each displacement level could be captured by the FE model. However, analytical results exhibit fatter hysteresis loops than experimental results.

Fig.6.4.1-2 to Fig.6.4.1-4 show the principal stresses in the specimen at different stages, $0.5P_i, P_i, 	ext{ ductility } 3$. These figures demonstrate the changing course of stress flows during the loading course. Based on the stress flows, compressive
concrete struts in the strut-and-tie model can be drawn out. Fig. 6.4.1-2 shows that in the early loading stages, the precast wall performed as its monolithic prototype in which the diagonal concrete strut was the main load resistant component. This stage, the lateral load was directly transferred from the loading point to the opposite lower corner of the wall by the diagonal strut. The maximum compressive stress appeared in the compression corner of the wall.

With increasing load, the stress flows changed as shown in Fig. 6.4.1-3. A diagonal concrete strut developed in the lower wall panel, which takes part in resisting lateral load transferred from upper wall panel. The load paths changed from those in Fig. 6.4.1-2. From Fig. 6.4.1-3, the stress flow shows that the two wall panels behaved as independent squat walls which were revealed by diagonal stress flows.

When displacement ductility reached 3, as shown in Fig. 6.4.1-4, a diagonal stress flow started from midline of the topside of the wall foundation interface to the upper left corner of the lower wall panel appeared. This strut resisted the main part of the lateral force; as a result, concrete crushing in lower corner did not occur in this wall. Another point is the critical zone in lower wall panel. Referring to the stress distribution of horizontal reinforcement, it reveals that the horizontal component of main the diagonal compression force coming from upper wall panel is resisted by the horizontal bars at the end of this concrete strut. The horizontal bars in tension passing through this region reduce the strength of concrete in the nodal zone. Cracks with a high density were observed in this region. Later concrete spalling occurred under cyclic load, which further deteriorated the compression strength of the concrete in this region. As a result, the portion of lateral force resisted by the diagonal concrete strut reduced; on the other side, requirement on horizontal bars increased. Shown in Fig. 6.4.1-6, the stress in diagonal reinforcement...
in a horizontal connection is 372MPa as shown in Fig.6.4.1-7. Therefore, a predictable shear failure would happen at the upper end of the diagonal reinforcement embedded in the base beam as observed in the experiment.

Fig.6.4.1-1 Comparison between Analytical and Experimental Results

Fig.6.4.1-2 Principal Stress at 0.5 $P_f$
6.5 Simulation of Specimen W3

6.5.1 2-D Model

A. Meshing of Specimen W3

In simulating the behavior of specimen W3 under cyclic loads, a two-dimensional model was built as shown in Fig.6.5.1-1. In this model, line interface element CL24I with 3+3 node was applied to simulate the behavior of the gap between the upper and lower wall panels. Diagonal reinforcement in the horizontal connection was represented by embedded reinforcement elements which passed through the horizontal connection. Similar properties of material as those in specimen W2 are applied in this model.

![Fig.6.5.1-1 Mesh of Specimen W3](image)

6.5.2 Analytical Results

Comparison of the analytical and the experimental hysteresis loops was shown in Fig.6.5.2-1. It showed that the behavior of the specimen W3 could be predicted with good agreement including strength and unloading stiffness.
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The changes of stress flows in the specimen were shown in Fig.6.5.2-2 to Fig.6.5.2-4, which showed that the lateral load acting on the top beam was resisted by different mechanisms at different loading stages. Fig.6.5.2-2 showed the principal compressive stresses in the specimen W3 at load $-0.5P_t$. At this stage, the stresses flow revealed that one diagonal concrete strut connecting loading point and opposite lower wall corner was the main shear resisting component. It hints that at the early loading stage, the precast shear wall behaved as a monolithic wall. However, compared with stresses in specimen W2 in previous section, more even distribution of the principal stresses in specimen W3 exhibited. This was because more than one shear connectors were used in the horizontal connection.

With increasing load, the stress flow changed as shown in Fig.6.5.2-3. At same time, cracking patterns observed at this loading stage showed agreement with these stress flows. From these stress flows, the diagonal cracks in the lower wall panel could be expected, which means that at this stage the precast wall was working as two separated wall panels.

When the wall was further loaded with higher displacement level, in upper wall panel two main diagonal concrete struts served as the main shear resisting components as shown in Fig.6.5.2-4. These two diagonal concrete struts changed the internal load paths in the lower wall panel. It was also found in the lower wall panel that a diagonal concrete strut starting from mid-point of baseline which formed another load resisting component. As a result, the shear resisting requirement on the diagonal concrete strut connecting lower compression zone decreased. Therefore, concrete crushing was not observed in the lower corner of this wall. As in specimen W2, the critical region of specimen W3 was at region with distance 650mm from wall and foundation interface. This is because this region was the main zone to resist diagonal compression transferred from upper wall panel. As
observed in experiment, horizontal bars in tension passed through this region and cracks with a high density appeared. These factors decreased the compression of strength of the nodal zone of diagonal strut. Concrete spalling occurred in the late loading stage was observed. Therefore, referring to the stresses in Fig.6.5.2-5, shear failure could be expected. These features of specimen W3 were similar to those occurred in specimen W2. Therefore, the failure location could be predicted.

Fig.6.5.2-1 Comparison of Analytical and Experimental Result of Specimen W3

Fig.6.5.2-2 Principal Stress at -0.5 r;
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**Fig. 6.5.2-3 Principal Stress at** $\theta$

**Fig. 6.5.2-4 Principal Stress at ductility -3**

**Fig. 6.5.2-5 Principal Stress Horizontal Reinforcement at Ductility -3**
6.6 Simulation of Specimen W4

In this research I-Beams were used as flexural reinforcement in boundaries of specimen W4, W5 and W6. I-Beams were simulated by truss elements in 2-D and 3-d models and by curved shell elements in 3-D model.

6.6.1 2-D Model

In previous simulation of W1, the channel were modeled by truss elements, the analytical results showed good agreement with experimental results. In simulating the behavior of wall W4 with cross section shown in Fig.6.6.1-1, the I-Beam was modeled by three truss elements. The total sectional area of the three elements equaled to that of I-Beam, as shown in Fig.6.6.1-2. Two truss section areas were equal to that of flanges of I-Beam, one was equal to that of web of I-Beam. Specimen mesh W4 was shown in Fig. 6.6.1-3. Similar to walls W1, W2 and W3, different thicknesses were assigned to top beam, base beam and wall.

![Fig.6.6.1-1. Cross section of wall W4](image1)

![I-Beam Truss element](image2)

- Truss 1: Area of flange=685.3mm^2
- Truss 2: Area of web=679.4mm^2
- Total area=2050mm^2

![Fig.6.6.1-2. Simulation of I-Beam in 2-D model](image3)
6.6.2 Analytical Results

The analytical result was shown in Fig.6.6.2-1. It can be seen from this figure that the predicted ultimate strength was much lower than the experimental result. However, the configuration of I-Beam was different from that of channels, which exhibited a clear influence on the strength of walls. In the plane of wall, confining effect of I-Beam on concrete, shown in Fig.6.6.2-2, should be taken into account. 2-D models were built to investigate confining effect of the I-Beam on the encased concrete. Different reinforcement ratio of confinement had been used in the simulation, but as suggest by Kwan (2001), the increment of shear capacity of the wall due to confinement had an upper limit, that is, beyond certain limit, further increase of the concrete confinement may not produce any effect. The analytical result was compared with the experimental results as shown in Fig.6.6.2-3. In the FE analysis, a high confinement ratio is 3.2% was used. The largest capacity of wall with additional confinement was about 667kN, while the experimental result was about 785kN, the rate of the analytical result to experimental result was about 0.85, which means that there were other factors affecting the strength of the specimen besides the confining effect of the I-Beams.
Fig. 6.6.2-1 Comparison of Analytical and Experimental Results of W4 by 2-D Model

Fig. 6.6.2-2. Confinement Effect of I-Beam
6.6.3 3-D Model

A. Confining Effect

A 3-D model of specimen W4 was developed with the same material models in previous sections, particularly, in which a high confinement ratio 3.2% was used to simulate the confining effect of I-Beam on the concrete, as shown in Fig.6.6.3-1. However, as shown in Fig.6.6.3-2, with such a high confinement ratio, the shear capacity of the model was similar to that in 2-D about 658kN, thought ductility of model increased. However, the effects of the embedded I-Beam on the behavior of wall in this experiment were not fully captured.

Degree of the agreement analytical results of wall with Channels and that with I-Beam to their own experimental result was different as shown above. Influence of the different confinement ratio on the strength was also investigated and the analytical result was about 15% lower than experimental result. Therefore, besides the confining effect of I-Beam, there were some other helpful factors. In loading
process, the web of the I-Beam in the walls was in a state of plane stress shown in Fig.6.6.3-3, which made the I-Beam with the encased concrete as a dowel or short column. This improved the strength of the walls.

To investigate the effects of plane stress of the web of the I-Beam on the strength of the wall, firstly, concrete was simulated by brick elements and the 2-node beam elements with same sectional properties of the I-Beam were used to simulate the true I-Beams, the wall usually exhibited compression failure of concrete at lower corner as it just passed the yielding point. That means this kind of element could not account for the confinement effect of I-Beam.
B. Curved Shell Element

To simulate true behaviors of I-beam, namely, confining effect on the encased concrete and plane stress in web of I-Beam, curved shell elements were selected as shown in Fig.6.6.3-4. In the simulations, an equivalent I-Beam was used. The thickness and length of flange of I-Beam were assumed equal to 5.1mm and 140mm, which made 3-D model of W4 easy to be built with same sectional area and similar second moment of area to actual I-Beam. Thickness and width of web are not changed. The web elements used 4-node element-Q20sh, and the flange elements used 8-node element-CQ40s as shown in Fig.6.6.3-4 and Fig.6.6.3-5.
The curved shell elements in DIANA are based on isoperimetric degenerated solid approach by introducing two shell hypotheses:

- It assumes that normals remain straight, but not necessarily normal to the reference surface. Transverse shear deformation is included according to the Mindlin–Reissner theory.
- It assumes that the normal stress component in the normal direction of a lamina basis is zero: $\sigma_{n n(I,n)} = 0$. The element tangent plane is spanned by a lamina basis which corresponds to a local Cartesian coordinate system $(x, y)$ defined at each point of the shell with $x$ and $y$ tangent to the $\xi$, $\eta$ plane and $z$ perpendicular to it.
- The in-plane lamina strains $e_{xx}$, $e_{yy}$ and $\gamma_{xy}$ vary linearly in the thickness direction. The transverse shear strains $\gamma_{xz}$ and $\gamma_{yz}$ are constant in the thickness direction. Since the actual transverse shearing stresses and strains vary parabolically over the thickness, the shearing strains are an equivalent constant strain on a corresponding area.
- They must be thin, i.e., the thickness $t$ must be small in relation to the dimensions $b$ in the plane of the element.

As shown in Fig.6.6.3-6, the analytical shear capacity was approximately equal to the experimental result. It hints the configuration effect of I-Beam increased the strength of wall W4 about 19%. However, the hysteresis loops of the FE analysis was fatter than the experimental loops, which means that the capacity of the energy dissipation was not predicted accurately. Fig.6.6.3-7, 8 and 9 showed that the critical features of the behavior of the wall can be simulated with a reasonable agreement.
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Fig. 6.6.3-4: Curved shell elements, characteristics

Fig. 6.6.3-5: Models of I-Beam and details of elements
Fig. 6.6.3-6 Comparison between Analytical and Experimental Results of W4

Fig. 6.6.3-7 Comparison failure mode
C. Coulomb Friction Model

(1) Linear Coulomb Friction Model

During testing of W4, sound of friction was heard, this means that slippage
between the I-Beams and the concrete in the central part of the wall occurred. In Fig. 6.6.3-9, the spalling of concrete along the inner boundary of the I-Beams which decreased the strength of the compression struts, as a result, decreased the strength of the wall in the last cycles of test.

During loading test, the concrete struts resisted diagonal compression which generated a component perpendicular to the flanges of the I-Beams in the wall plane. Therefore, bond-slip material model was not the suitable one to model the behavior of the interaction between the I-Beams and the concrete.

To simulate the true interaction of the I-beams and concrete, interface elements with Coulomb friction material model were introduced into the model, as shown in Fig.6.6.3-10. As shown in Fig.6.6.3-11 and 12, CQ48I element, a type of structural interface element, was used which was an interface element between two planes in a three-dimensional configuration. The structural interface elements described the interface behavior in terms of a relation between the normal and shear tractions and the normal and shear relative displacements across the interface. The formulation of the plane interface elements was fully isoparametric, which means that the plane interface elements may be flat as well as curved.

As shown in Fig. 6.6.3-13, the models with and without interface elements were compared. The linear material model was applied in the interface elements. Fig.6.6.3-13 also showed that strength of the models with and without interface elements were approximately similar, but the reloading stiffness of models with interface elements was lower than that without in positive and negative directions, which consequently influenced capacity of energy dissipation of model. However, from Fig.6.6.3-14, it is shown that capacity of energy dissipation was not predicted exactly.
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\[ D_{\epsilon} = 0.43E_s = 0.43 \times 29000 = 12470 \text{ N/mm}^2 \]
\[ D_{\sigma} = 29000 \text{ N/mm}^2 \]

(a) elevation view  
(b) perspective view

Fig. 6.6.3-10 3-D model with interface elements

(a) topology  
(b) displacements

Fig. 6.6.3-11. Characteristics of CQ48I

(a) displacements  
(b) relative displacements  
(c) tractions

Fig. 6.6.3-12 Variables of three-dimensional structural interfaces
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Fig. 6.6.3-13 Comparison between model with and without interface elements
(Interface element with linear material model is used)

Fig. 6.6.3-14 Comparison between experimental results and analytical one with interface element with linear material model

(2) Nonlinear Coulomb Friction Model

- Analytical Results
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Based on previous information, in this study, different friction angles and dilation angles were selected to investigate these parameters' effect on the behavior of wall W4 as shown in Appendix A. Except the one with dilation angle equal to zero, others showed agreement with experimental results, including strength, energy dissipation, unloading and reloading stiffness. The result of 3-D models considering the nonlinear frictional material model and the gap generation were shown in Fig. 6.6.3-15. From Fig. 6.6.3-15 it can conclude that strength, loading and unloading stiffness of the FE analytical results were similar to those of experimental result. Pinching of hysteresis loops was accurately captured and overall capacity of energy dissipation was predicted with good agreement.

![Graph comparing experimental and analytical results](image)

**Fig. 6.6.3-15 Comparison between Experimental Results and Analytical Ones with Interface Element of Nonlinear Frictional Model**

6.6.4 Summary

In this part of the FE analysis, works focused on simulation of wall W4. Based on previously successful simulation of wall with channels, 2-D models were developed to study behavior of wall W4 with I-Beams. The I-Beams were substituted by a set of equivalent reinforcement with total sectional area equal to that of I-Beam. However, the lateral load-resisting capacity was 15% lower than
experimental one. A similar 3-D model was built with equivalent reinforcement, which showed similar shear capacity to that of 2-D model. The confining effect of the I-Beam encased concrete was also taken into account in the two previous models by additional confinement. Considering the plane stress state of the web of the I-Beams in the loading process, two kinds of shell elements were introduced into the 3-D model as an equivalent I-Beam. In this model, the shear capacity showed good agreement with experimental one, except with much higher capacity energy dissipation. In order to simulate the true interaction behavior between the I-Beams and the concrete, interface elements with linear and nonlinear Coulomb friction material were used in this study. Different frictional angle and dilation angles were applied to investigate their influence on the behavior of the specimen. In simulations with nonlinear friction material model, a friction angle and dilation angle showed their critical influence on the stability of calculation.

6.7 Simulation of Specimen W5

6.7.1 Analytical Model

Three-dimensional model was built to simulate the behavior of specimen W5, as shown in Fig.6.7.1-1, in which solid elements were used to simulate concrete and truss models were applied to represent vertical and horizontal reinforcements, two kinds of shell elements as used in simulation of specimen W4 were used to simulate the behavior of the I-Beams as shown in Fig.6.7.1-2, the behavior of interfaces between the I-Beams and the surrounding concrete and interfaces between upper and lower wall panels were modeled by interface elements with different material model. Regarding the tolerance of the diameter of the bolt holes and bolts, similar strain stress curve for those steel plates as shown in Fig.6.4.1-2 are applied.
6.7.2 Analytical Results

The comparison between the analytical and the experimental results was shown in Fig.6.7.2-1. From this figure, it showed that the ultimate strength was captured accurately. However, the initial stiffness of analytical result was higher than that of experimental result. After specimen W5 was unloaded to zero, the residual deformation of analytical result was higher than that of experimental result. This
may be due to the smaller predicted shear deformation.

Fig. 6.7.2-2 to Fig. 6.7.2-4 show the changes of the principal compression stresses during the loading course. Similar to specimen W2, in the early loading stage, the precast specimen W5 exhibited as a monolithic wall in which the diagonal concrete strut connecting loading point and opposite lower corner acted as main load resisting mechanism as shown in Fig.6.7.2-2. With increasing load, the internal load paths changed to those shown in Fig.6.7.2-3, in which the diagonal compression concrete struts formed in each wall panel. These diagonal concrete struts served as the main resisting components to the lateral load acting on the top beam. When the displacement ductility reached -3, one new diagonal concrete strut starting near the mid-point of the interface of wall and foundation formed as shown in Fig.6.7.2-4. The contribution of this diagonal concrete strut reduced requirement on strength of the diagonal concrete strut starting from the lower corner of the specimen. Dense cracks were observed at a location with a distance 650mm from base of the wall, which decreased the compression strength of concrete in this region. Referring to the stresses of horizontal bars shown in Fig.6.7.2-5, the horizontal bars passing through this nodal zone yielded, which means that large plastic deformation was allowed. As a result, the compression strength of concrete in this zone decreased. Concrete spalling appeared at this loading stage deteriorated the damage. Shear failure at this location could be expected.
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Fig. 6.7.2-1 Comparison between Analytical and Experimental Results

Fig. 6.7.2-2 Principal Stresses of Specimen W5 at -0.5 $p_f$
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Fig. 6.7.2-3 Principal Stresses of Specimen W5 at $\eta$

Fig. 6.7.2-4 Principal Stresses of Specimen W5 at ductility -3
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6.7.2 Summary

In this part of FE analysis, 3-D model was built to simulate the behavior of specimen W5. Interface elements were used between the upper and lower wall panels. The interaction between the I-beams and the concrete was captured through interface elements. Similar to those in W2, the properties of embedded element and steel plates were model for the connection. The analytical result showed good agreement with experimental result including strength, stiffness and ultimate displacement except that the analytical hysteresis loops were fatter than the experimental loops. The appearing sequence of cracks was well simulated and the failure modes and failure location agreed with that observed in experiment.

6.8 Simulation of Specimen W6

6.8.1 Analytical Model

Three-dimensional model was built to simulate the behavior of specimen W6, as shown in Fig.6.8.1-1. The difference between mesh of specimen W6 and W5 was the placement of diagonal reinforcement in the horizontal connection. Material
models for the concrete and steels were referring the tested result. The behavior of the interfaces between the I-Beams and the surrounding concrete and the interfaces between the upper and lower wall panels were simulated by the interface elements with different material model. Regarding to the tolerance of diameter of bolt holes and bolts, similar strain stress curve for these steel plates as shown in Fig.6.4.1-2 were applied.

Fig.6.8.2-1 Principal Stresses of Specimen W6 at -0.5 P
6.8.2 Analytical Results

Fig. 6.8.2-1 Comparison between Analytical and Experimental Results of W6

Fig. 6.8.2-2 Principal Stresses of Specimen W5 at -0.5 \( \tau \)
Fig. 6.8.2-2 Principal Stresses of Specimen W5 at - $p_1$

Fig. 6.8.2-2 Principal Stresses of Specimen W5 at Ductility -3
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6.9 Summary

In this chapter, to provide insight into the behavior of these specimens and investigate critical parameters, finite element models were built to simulate the behavior of these specimens. The following can be concluded from finite element analysis:

1. Behavior of specimens W1, W2 and W3 could be accurately predicted by 2-D models in which reinforcement was modeled by truss elements and concrete was simulated by plane stress elements. 3-D models were also built to simulate W1, W2 and W3 which had similar results to the 2-D model, which indicated the behavior of channels used in W1, W2 and W3 can be accurately simulated by truss elements.

2. Analytical results of W4, W5 and W6 by 2-D models similar to those for specimens with channels exhibited much lower strengths than the experimental results. This situation was also observed when W4, W5 and W6 were modeled by 3-D models in which the I-Beams were replaced by truss elements.

3. To understand the beneficial effects of the I-Beams on the performance of
walls, 3-D models with high confinement reinforcement ratio were also studied. These models exhibited higher ductility than those without additional confinement. But no increase in strength is observed.

4. In 3-D models with shell elements, the interaction between the I-Beams and the surrounding concrete was studied by interface elements of friction material. The behavior of the interface between the upper and lower wall panels was also modeled by interface elements. The analytical results show that the main features of the behavior of specimens with I-Beams can be predicted with good agreement.
CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

To investigate the seismic performance of precast segmental shear walls with horizontal connections a total of six specimens were designed and tested under cyclic load to failure, two of the shear walls were cast monolithically as prototypes. Steel sections were used in boundaries of these walls to serve as flexural reinforcement. One of the walls had channels and the other had I beams for the flexural reinforcement. For each of these wall types, two precast variations were tested. Parameter investigated included the horizontal connections and number of shear connectors. Seismic characteristics including strength, stiffness, pinching effect and energy dissipation were investigated.

To predict behavior of general precast shear walls with horizontal connection, a model based on macro model was develop in this research. A parametric study based on this model has been done including bolt types and location of connection.

Based on the crack patterns of the specimens that developed during the course of loading, strut-and-tie models were built to reveal internal load paths in these walls for different loading stages, the strut-and-tie models elucidate changes of mechanisms of load transfer in different loading stages.

As a powerful tool in engineering analysis, finite element analysis (FEA) was undertaken to study the performance of these specimens under cyclic loads. Analytical results were compared to the experimental ones.

Conclusions that can be drawn from this research include:
1. All precast walls except W6 showed good performance in comparison with their monolithic prototypes including strength, stiffness and capacity of energy dissipation. W6 failed with shear failure of connecting bolts in the flexural connection.

2. The precast shear walls exhibited similar yielding strength, ultimate strength, and drift level as their own monolithic prototypes. However, these precast shear walls experienced a little lower stiffness and capacity of energy dissipation capacity compared to their prototypes.

3. The yield displacement of the precast specimens was postponed compared to that of their prototypes because of additional deformation due to tolerance in the bolted connections, while the ultimate displacement of precast walls occurred earlier.

4. The ductility of precast walls was lower than that of their prototypes. One source was the built-in tolerance of bolts relative to the wholes, which caused yielding to occur at a larger displacement, while ultimate displacement occurred earlier.

5. Except for the horizontal connection in specimen W6 that failed, the others successfully transferred the lateral load to the lower wall panel. As a result, the tests demonstrated that horizontal connections can provide sufficient strength, also accelerates construction speed, and decrease the labor on site.

6. In early loading stages, the internal load paths in the precast walls were similar to those in monolithic walls, whereas, with increasing displacement level, the internal load paths in the precast walls diverged from those of their monolithic prototypes. Cracking patterns in the precast walls were different from those of the corresponding monolithic walls. The inclined angle of cracks in precast walls was lower than that of the monolithic walls.

7. The difference of internal load paths in the precast walls and their monolithic prototypes in later loading stages caused different failure modes and failure locations of these walls.

8. In later loading cycles, the upper panels of the precast walls behaved as squat shear walls in which two diagonal concrete struts formed, which influenced the force paths in the lower wall panels.
9. It was found that, compared to walls with channels, walls with I-beams exhibited higher strength when accounting for the sectional area of the steel section.

10. Two-dimensional and three-dimensional models were applied to simulate the behaviors of specimen W1 and W4. It was found that the channels could be modeled by truss elements with equal sectional area with a two-dimensional model, which produced similar performance to that of a three-dimensional model and to the wall. But, walls with I-Beams in which the I-Beams were represented by truss elements with equal sectional area in a two-dimensional model led to fifteen percent lower strength than the experimental results.

11. Both monolithic shear walls exhibited flexural failure with concrete crushing in the bottom corner of the specimens while their precast counterparts experienced shear failure with fracture of the horizontal reinforcement.

12. Finite element analysis using DIANA provided a good prediction of the nonlinear behavior of the monolithic and precast shear walls including the load-displacement hysteretic loops and the displacement ductility.

13. The internal stress flows predicted by the FE analysis can be utilized to develop strut-and-tie model which can be used as tool to understand the internal force transfer mechanism in reinforced concrete structures or members. Principal compression stress distribution at early loading stages indicated the shear force was mainly resisted by a diagonal strut. With increasing numbers of cycles of loading the strut angle changed.

14. In specimen design, to effectively transfer the force from the upper panel to the lower panel using a simple connection, the shear and bending moment at the connection were separated and carried by different parts of the connection. This gives the designer a clear strength design and unambiguous force transfer for which to provide. The connections used in the design were mechanical ones which enable on-site assembly of the precast wall with simple braces as for steel structures.
15. The number of shear connectors in the connection had an appreciable effect on the yield displacement, ultimate displacement, and stiffness. With more shear connectors, the stiffness of the precast walls increased.

16. Diagonal reinforcement placed near the base of the walls influenced the performance of the wall under cyclic loads. In contrast to the requirements of the ACI 318 codes that diagonal reinforcement must passing through any section within the height of the less of $0.5h_w$ or $0.5l_w$, diagonal bars were placed only within a height $0.31H$ of the monolithic walls above the base section, which successfully prevented the walls from sliding shear failure near their bases.

17. The developed model for precast walls with horizontal connections can be used to predict the lateral-displacement, which produced satisfactory agreement with experimental results, including stiffness, yielding displacement, yielding strength and ultimate strength. Parametric study based on this model indicates that high strength friction bolt is preferred in precast wall with bolt connections. And stronger material or bigger section should be used to restrain the deformation of connection.

### 7.2 Recommendations

Based on the present research including experimental results and FEM analysis, a series of design recommendations are listed out as following:

1. In design precast shear walls with horizontal connections, it is beneficial to separate the shear force and moment at the connection, which are separately resisted by independent components of the connection, i.e., flexural moment is resisted by flexural-resistant components at two ends of the wall, and shear is taken by the shear-resistant components at central (web) zone of the connection. This will give designer a clear concept of load transfer in the connection.
2. Connection design should be based on the clear sectional area of the steel section and actual forces at the connection. Local failure of the connection in steel structural should be prevented by taking account for the overstrength of the steel reinforcement in the joining walls and preventing large deformation occurred in the connection which causes loss of the compatibility between the shear and flexural resistant components. Another factor affecting the strength of wall is the enhancing effect of I-beam on the strength of the walls with embedded I sections. 20 percent of design strength of the walls, considered as this effect on strength, is reasonable to be added to the design strength. As a result, in connection design, three major factors, overstrength of reinforcement in wall panels, enhancing effect of I section and limitation of deformation in connection, should be satisfied at same time.

3. More than one shear connectors can be used to resist the shear force at the connection, which is of advantage in distributing the force into a larger area and is also beneficial to prevent the local failure of the wall panel if only one connector to take a large magnitude of the forces.

4. To arrest the deformation of connection due to the tolerance between bolts and bolt holes, high strength friction bolts, compulsorily used in steel structure under vibration or other dynamic situations, are preferable to be used in the steel connections. High tensile strength bolts with fitted bolt holes can be an alternative. To reduce the deformation of the steel member at the connection, doubler plates can be welded to the ends of the members to increase the net cross sectional area which is reduced by the bolt holes.

5. In the design squat wall, especially with higher reinforcement ratio at ends of the wall which may typically fail in sliding shear under cyclic load, diagonal reinforcement can be a good choice to avoid this kind of failure mode known as brittle failure and meantime increase the ductility of the walls to enhance the chance of survival of structures in earthquakes of high magnitude.

6. Fine steel mesh will reduce deterioration of the crushing of concrete that occurs at the end of the diagonal bars in the lower wall panel. As shown in the strut-and-tie models developed for the precast walls, this region is TTS.
node where the strength of the strut is reduced. Fine steel mesh will take part of the tension forces, therefore, detains the cracks, reduce the strain in the concrete and suspend the crushing, finally increases the strength in the later loading stages and also increases ductility of walls.

7. It is better to provide confinement in the lower 1/3 height of the wall if monolithic walls will be used in strong earthquake regions. The confinement reinforcement will improve displacement ductility of the monolithic walls.

8. In squat walls, the flexural moments is not critical compared with the shear force due to its stocky configuration. Sliding shear failure is commonly observed under cyclic loads. Diagonal reinforcement placed in the bottom of the wall is a better option in preventing sliding shear failure of the squat walls under cyclic loads, especially in walls with concentrated reinforcement at two ends. It will increase the shear strength and ductility of the walls if properly designed.

9. Strut-and-models based on clear load paths will help understand the actual behavior of the walls and therefore can guide the shear wall design and prevent the brittle failure in the future practical application of the walls.

10. The walls with steel sections acting as flexural reinforcement can be designed in the same way as in the design of normal reinforced concrete shear walls. The stiffness of the precast walls with the horizontal connections used in present research is about 60 percent of their monolithic walls.

11. For walls designed under cyclic loading, deformed bars is preferred than plain bars, which will increase the interaction between reinforcement and surrounding concrete and decrease the relative slip between the two materials during the course of loading. As a result, the hysteresis loops will be fattened, which means higher capacity of energy dissipation.

12. The suggested model based on the macro model can be useful tools in actual design and theoretical analysis with properly setting material models.

13. High strong friction bolt or weld is preferred in precast walls with bolt connection. Stronger material or section with bigger sectional area should be used to satisfy the strength requirements and deformation limits.
Chapter 7 Conclusions and Recommendations

7.3 Future Works

Based on the present extensive research work, experiment and FEM analysis, a series of future study can be done to further understand the performance of the precast walls with horizontal connections.

1. In present study, the vertical reinforcement was discontinuous at the horizontal connection, which caused apparent deformation in the connection and thus influenced the stiffness of the precast walls and the internal load transfer paths inside the precast walls. Couplers and other lapping devices or methods may be applied in the horizontal connections, which may reduce the deformation of the connections, especially the sliding along the connection, and increase the stiffness of the precast wall and change the internal load paths.

2. The locations of the horizontal connection can be variable, for example, at the interface of wall-foundation. For moderate and high rise building, the gravity load of upper structures will exert much influence on the behavior of the lower walls in ductility, stiffness and strength. Thus, the gravity load from upper levels can also be added in the future test.

3. In this study, one the precast wall failed in connection, which hints that in designing precast walls the strength of the connection shall be ensured to make the wall reach its design goals without failure. And other different types of the connection can be tested in the future work to optimize the performance of the precast wall.

4. Shear walls with different configurations, for example, barbell walls and wall with openings, are widely used in practical application. The performance of these kinds of walls with embedded steel shapes has not been extensively studied. More research work can be done in the future to accurately their behavior and wide their use in practical structures.

5. Effects of confinement on the performance of the precast shear walls have not been fully investigated, which may improve the ductility of walls that will fail in crushing concrete in compression at the extreme fiber.
6. A widely parametric study by FEN shall be applied to study the influence of the different factors on the local and integral properties of these walls. It includes gravity load from upper levels equal to 0.1 and 0.2 of the uniaxial compressive strength of the wall cross section as in medium-height and high-rise buildings, respectively, different reinforcement ratio in horizontal and vertical directions, and different sectional areas of the embedded steel shapes. The locations of the horizontal connection, for example at the interface of wall-foundation interface, and the wall with local strengthening reinforcement, especially at the failure connection in the lower panels, shall be factors to be investigated in the future FEM study. In the present study, most precast walls failed in the connection zone in the lower wall panel. Therefore, different measures can be applied in the FEM analysis to find out the solutions to prevent this kind of failure.

7. The sectional effect of I-beam in increasing strength and stiffness of the wall was observed and successfully modeled by FEM, an extensive parametric study shall be done in the future on the configurations of steel shapes and steel ratio of I-beam to the wall which will influence the behavior of the walls.

8. In this research, the suggested model ignored the prediction on the load vs. displacement relation before cracking of the wall. Tension stiffening effect from surrounding concrete in between cracks was not taken into account, which may influence the load-displacement curve. And more refined spring model may produce better prediction. All these points on the suggested model can be done in the future work.
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Reference


APPENDIX

In simulating behavior of specimens with I-beams, different frictional material properties of interface elements have been tested. Different friction angles and dilation angles were applied and showed critical influence on the stability of calculation.

<table>
<thead>
<tr>
<th>Fig. No.</th>
<th>Friction angle (degrees)</th>
<th>Dilation angle (degrees)</th>
<th>Interaction after gapping</th>
</tr>
</thead>
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<tr>
<td>Fig A1.</td>
<td>30</td>
<td>12</td>
<td>Aggregate interlocking</td>
</tr>
<tr>
<td>Fig A2.</td>
<td>20</td>
<td>12</td>
<td>Aggregate interlocking</td>
</tr>
<tr>
<td>Fig A3.</td>
<td>20</td>
<td>12</td>
<td>Brittle cracking</td>
</tr>
<tr>
<td>Fig A4.</td>
<td>17</td>
<td>12</td>
<td>Brittle cracking</td>
</tr>
<tr>
<td>Fig A5.</td>
<td>17</td>
<td>0</td>
<td>Brittle cracking</td>
</tr>
<tr>
<td>Fig A6.</td>
<td>12</td>
<td>12</td>
<td>Brittle cracking</td>
</tr>
<tr>
<td>Fig A7.</td>
<td>12</td>
<td>9</td>
<td>Brittle cracking</td>
</tr>
</tbody>
</table>

Fig A1. Comparison between analytical and experimental results.

$\phi = 30^\circ, \varphi = 12^\circ$ Considering aggregate interlock after gap
Fig A2. Comparison between analytical and experimental results.
\[ \phi = 20^\circ, \varphi = 12^\circ \] Considering aggregate interlock after gap.

Fig A3. Comparison between analytical and experimental results.
\[ \phi = 20^\circ, \varphi = 12^\circ \]
Fig A4. Comparison between analytical and experimental results.

$\phi = 17^\circ$, $\varphi = 6^\circ$

Fig A5. Comparison between analytical and experimental results.

$\phi = 17^\circ$, $\varphi = 0^\circ$
Fig A6. Comparison between analytical and experimental results.

\[ \phi = 12^\circ, \ \varphi = 12^\circ \]

Fig A7. Comparison between analytical and experimental results.

\[ \phi = 12^\circ, \ \varphi = 9^\circ \]