Damage assessment of reinforced concrete frames to ground excitations

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DAMAGE ASSESSMENT OF REINFORCED CONCRETE FRAMES TO GROUND EXCITATIONS

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2005

Damage Assessment of Reinforced Concrete Frames to Ground Excitations

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School of Civil & Environmental Engineering

A thesis submitted to the Nanyang Technological University in
fulfillment of the requirements for the degree of Doctor of
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2005

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TABLE OF CONTENTS

ACKNOWLEDGMENT	i
TABLE OF CONTENTSS	ii
SUMMARY	vi
LIST OF TABLES	viii
LIST OF FIGURES	viii
LIST OF SYMBOLS	xii
1 INTRODUCTION	1
1.1 BACKGROUND	1
1.2 PURPOSE AND SCOPE	3
1.2.1 RESPONSE ANALYSIS	3
1.2.2 DAMAGE ASSESSMENT	5
1.2.3 SCOPE	6
2 EXPLOSION-INDUCED GROUND MOTIONS (EIGMS)	7
2.1 CURRENT EIGM PROVISIONS	8
2.2 RESPONSE PHASES	8
2.3 IMPLICATIONS	9
3 LITERATURE REVIEW - PHASE I OF EIGM	12
3.1 GENERAL EIGM OBSERVATIONS	12
3.2 APPROXIMATE RESPONSE ANALYSIS	13
3.3 STRUCTURAL DAMPING	15
3.4 BEAM THEORIES	15
3.4.1 EULER-BERNOULLI BEAM THEORY (EBT)	16
3.4.2 TIMOSHENKO BEAM THEORY (TBT)	17
3.4.3 DYNAMIC ANALYSIS - TBT vs. EBT	19
3.4.4 FINITE ELEMENT METHOD (FEM)	20
3.5 STRAIN RATE EFFECT	24
3.5.1 CLASSICAL EQUATION: MALVERN	24
3.5.2 CONCRETE COMPRESSIVE STRENGTH-ENHANCEMENT	25
3.5.3 CONCRETE TENSILE STRENGTH-ENHANCEMENT	26

3.5.4	STEEL REINFORCEMENT STRENGTH-ENHANCEMENT	28
3.6	DIRECT SHEAR	28
3.6.1	SHEAR HINGE	29
3.6.2	DIRECT SHEAR CAPACITY	30
3.7	DAMAGE ASSESSMENT	30
3.8	IMPLICATIONS	31
4	LITERATURE REVIEW – PHASE II OF EIGM	33
4.1	MEMBER STRENGTH	33
	Shear Strength of Priestley et al (1994)	34
4.2	JOINT STRENGTH	35
4.2.1	JOINT ANALYSIS	38
4.2.2	JOINT EXPERIMENTATION	41
4.2.3	JOINT STRENGTH	42
4.3	FAILURE	43
4.3.1	MEMBER FAILURE (CHUNG ET AL, 1987)	44
4.3.2	JOINT FAILURE	45
4.4	POST-ELASTIC BEHAVIOR	46
4.4.1	THREE-PARAMETER MODEL OF PARK AND ANG (1985)	48
4.4.2	JOINT POST-ELASTIC BEHAVIOR	51
4.5	DAMAGE ASSESSMENT	53
4.5.1	DAMAGE ASSESSMENT: PROCESS	53
4.5.2	DAMAGE ASSESSMENT: TYPES	54
4.6	IMPLICATIONS	63
4.6.1	JOINT DIAGONAL SHEAR CAPACITY	63
4.6.2	DAMAGE ASSESSMENT	63
4.6.3	JOINT FAILURE	64
5	TIMOSHENKO BEAM VS. EULER-BERNOULLI BEAM	65
5.1	ANALYTICAL MODEL AND EIGM USED	65
5.2	FIXED-FIXED COLUMN	67
5.2.1	MODE SHAPES	68
5.2.2	TBT-19 vs. EBT-19	70
5.2.3	EFFECT OF NUMBER OF ELEMENTS USED IN EBT-FEM	77

5.2.4	DISCUSSION	79
5.3	FIXED-FREE COLUMN	80
5.3.1	MODE SHAPES	80
5.3.2	TBT-19 vs. EBT-19	83
5.4	EFFECT OF BOUNDARY CONDITIONS	87
5.5	DISCUSSION	89
6	PROPOSED METHODS AND TECHNIQUES – PHASE I OF EIGM	91
6.1	PROPOSED APPROXIMATE METHOD FOR RESPONSE ANALYSIS	91
6.1.1	PROPOSED APPROXIMATE METHOD FOR A SDOF SYSTEM	92
6.1.2	PROPOSED APPROXIMATE METHOD: MDOF SYSTEM	106
6.1.3	MDOF SYSTEM UNDER EIGM	107
6.1.4	PROPOSED APPROXIMATE METHOD: CONCLUSIONS	113
6.2	PROPOSED DAMAGE ASSESSMENT FOR PHASE I	114
6.2.1	PROPOSED DAMAGE ASSESSMENT PROCEDURE	115
6.2.2	DAMAGE ASSESSMENT: FIXED-FIXED COLUMN	120
6.2.3	CASE STUDY: DIAGONAL SHEAR FAILURE	125
6.2.4	CASE STUDY: DIRECT SHEAR FAILURE	130
6.2.5	DISCUSSIONS	131
6.3	PROPOSED SIMPLIFIED SINGLE COLUMN ANALYSIS	133
6.3.1	SCALING OF EIGMs	133
6.3.2	DAMAGE ASSESSMENT USING MATERIAL-LEVEL MODELING	136
6.3.3	FEASIBILITY OF APPLYING THE PROPOSED SIMPLIFIED METHODOLOGY	140
6.4	SUMMARY	147
7	DAMAGE ASSESSMENT OF A 6-STOREY RC FRAME – PHASE II OF EIGM	149
7.1	FRAMEWORK OF DAMAGE ASSESSMENT FOR PHASE II	149
7.2	FRAME DETAILS	150
7.2.1	MODE SHAPES AND MODAL FREQUENCIES	150
7.3	RESPONSE ANALYSIS – GLOBAL MODES	153
7.4	DISCUSSIONS	161

8	DAMAGE ASSESSMENT OF A 6-STOREY RC FRAME –PHASE I AND PHASE II OF EIGM	162
8.1	PROCEDURE AND CONSIDERATIONS	162
8.2	DAMAGE ASSESSMENT FOR PHASE I	163
8.3	DAMAGE ASESSMENT FOR PHASE II	165
8.3.1	GLOBAL DAMAGE ASSESSMENT	165
8.3.2	COLUMN SHEAR STRESS	166
8.3.3	COLUMN BENDING MOMENT	167
8.3.4	INTER-STOREY DRIFT RATIO AND JOINT FAILURE	169
8.3.5	COMBINED FAILURE ZONES FOR PHASE II	170
8.4	COMBINING PHASE I AND PHASE II RESULTS	172
8.5	COMPARISON: 2-STOREY AND 6-STOREY RC FRAMES	177
8.6	DISCUSSIONS	178
9	CONCLUSIONS AND RECOMMENDATIONS	181
9.1	CONCLUSIONS	181
9.1.1	RESPONSE ANALYSIS	183
9.1.2	DAMAGE ASSESSMENT	185
9.1.3	FRAMEWORK FOR DAMAGE ASSESSMENT	189
9.2	RECOMMENDATIONS	192
10	REFERENCES	193

SUMMARY

Within a multitude of hazards that can lead to structural failure, explosion-induced ground motions (EIGMs) is one. When subjected to EIGMs, the structure experiences small structural deformations and large induced shear forces during the loading duration (Phase I). After the loading duration, larger structural displacements are experienced during the free vibration duration (Phase II). While the local mode response is significant during Phase I, this is less significant during Phase II. For reinforced concrete (RC) frames subjected to EIGMs, research is conducted related to the response analysis and damage assessment.

During Phase I, the numerical examples have shown that Euler-Bernoulli elements can produce accurate response results as compared to Timoshenko elements. This is true when a sufficient number of elements is used, and the mass of the column is distributed at intermediate nodes. During Phase I, the shear forces and bending moment induced are the largest at the column base. This is regardless of the boundary condition at the top end of the column as demonstrated using a fixed-fixed and a fixed-free column.

An approximate method is proposed for the response analysis during Phase I. The approximate method is developed for a system of multiple degrees of freedom so that local mode response can be included. Furthermore, it produces a time history for the response, which is important to capture the peak forces during Phase I that can occur during the loading duration. The proposed approximate method produces accurate response results during Phase I. Further, the approximate response obtained at the end of Phase I can serve as the initial conditions for the response analysis of Phase II. In short, the approximate method can lead to substantial savings in computational effort by separating response analyses of Phase I and Phase II.

For the damage assessment, research is conducted for damage assessment during Phase I and Phase II. In this thesis, it has been shown that the shear strength of an

RC section may increase by about 40% under EIGM loading. A damage assessment technique is proposed for response of RC frames during Phase I. This technique considers the strength-enhancement effects and reflects the possible failure mechanisms. This is applied for a structural-level model, and the results are compared with those from a material-level model. The proposed damage assessment method produces damage results similar to those from a material-level model. In addition, the proposed damage assessment method can reflect the failure mode resulting from direct shear, which was not incorporated in the material-level model used for comparison. For the damage assessment during Phase II, a failure mode at beam-column joints has been incorporated via a limiting inter-storey drift ratio based on experimental observations. This joint failure mode has not been considered by other damage assessment techniques.

A simplified method for damage assessment of an RC frame has been proposed. The simplified method shows that the damage of an RC frame subjected to EIGMs can be determined by performing the damage assessment on the column at the first storey. Thus, the damage of an RC frame can be accurately assessed in a shorter time and requires a substantially less computational effort.

LIST OF TABLES

Table 3.1 - Shear and longitudinal wave speeds for stated materials	20
Table 4.1 - Maximum allowable joint stress	42
Table 4.2 - Typical range of values for hysteretic modeling	50
Table 4.3 - Seismic index and damage	56
Table 4.4 - Transformation of damage index to damage levels	63
Table 5.1 - Modal frequencies of fixed-fixed column	70
Table 5.2 - Summary of peak displacements and time of occurrence for EBT-19 and TBT-19 of fixed-fixed column	72
Table 5.3 -Summary of shear forces and bending moment for TBT-19 and EBT-19 models of fixed-fixed column	73
Table 5.4 - Shear forces and bending moments for EBT models of fixed-fixed column	78
Table 5.5- Modal frequencies of fixed-free column	81
Table 7.1 - Maximum absolute response of frame	153
Table 7.2 -Column damage indices for both phases	157
Table 8.1 – Shear failure during Phase I response	163

LIST OF FIGURES

Figure 2.1 - Extent of damage due to impulsive pressure loadings	7
Figure 2.2 – Illustration of Phase I and Phase II response	9
Figure 3.1 - Deformation for three beam theories (Wang et al, 2000)	16
Figure 3.2 - Frequency resonance dependence of beam deformation (Antes et al, 2004)	23
Figure 3.3 - Concrete compressive strength-enhancement (CEB, 1993)	26
Figure 3.4 - Observation and simulation of a column subjected to blast loading (Crawford et al, 2001)	31
Figure 4.1 - Statics of an interior beam-column joint	37
Figure 4.2 - Forces in a joint and joint models	40
Figure 4.3 - Failure modes in members (Chung et al, 1987)	45
Figure 4.4 - Hysteretic energy in elastic and inelastic systems	47

Figure 4.5 - Parameters in Park and Ang's three-parameter hysteretic model (Park and Ang, 1985)	51
Figure 4.6 - Illustration of MDRF (Roufaiel and Meyer, 1987)	58
Figure 4.7 - Cyclic loads of three different load sequence	59
Figure 5.1 - EIGM used and models of column used	67
Figure 5.2 - Selected mode shapes of TBT-19 for fixed-fixed column	69
Figure 5.3 - Displacement and rotation time histories for EBT-19 and TBT-19 of fixed-fixed column	71
Figure 5.4 - Induced shear forces and induced bending moments for EBT-19 and TBT-19 of fixed-fixed column	74
Figure 5.5 - Displacement profiles at maximum shear for fixed-fixed column	79
Figure 5.6 - Selected mode shapes for TBT-19 model of fixed-free column	82
Figure 5.7 - Displacement and rotation time histories for EBT-19 and TBT-19 for fixed-free column	85
Figure 5.8 - Induced shear forces and bending moment for EBT-19 and TBT-19 of fixed-free column	86
Figure 5.9 - Comparing induced shear and bending moment for fixed-fixed and fixed-free column	88
Figure 6.1 - SDOF system	93
Figure 6.2 - SDOF system and EIGM for study of structural damping	95
Figure 6.3 - General triangular ground acceleration	101
Figure 6.4 - Illustration of closed-form solution for SDOF system and one triangular EIGM	101
Figure 6.5 - Displacement and velocity correction factors	103
Figure 6.6 - Velocity and displacement response	104
Figure 6.7 - Effectiveness of using the approximate response at time $t = t_d$ as the initial conditions for Phase II response	106
Figure 6.8 - RC frame and EIGM used	109
Figure 6.9 - Approximate and exact deformations	111
Figure 6.10 - Induced shear and moment	112
Figure 6.11 - Flowchart for proposed damage assessment	119

Figure 6.12 - Response of fixed-fixed column including strength-enhancement effect using TBT-19 model	124
Figure 6.13 - Proposed damage index for fixed-fixed column during Phase I	125
Figure 6.14 - Induced shear force and bending moment for scaled EIGM	128
Figure 6.15 – Response of fixed-fixed column to scaled EIMG including strength enhancement effect	129
Figure 6.16 - Damage assessment for column under scaled EIGM	130
Figure 6.17 - Damage assessment of direct shear failure case	131
Figure 6.18 - Acceleration and velocity time histories for a triangular EIGM load	135
Figure 6.19 - Simulated EIGM at 50 m from explosion (Pan et al, 2004b)	138
Figure 6.20 - Failure mechanisms determined for 2-storey RC frame (Pan et al, 2004b)	140
Figure 6.21 - Flexural failure of fixed-fixed column under scaled EIGM	142
Figure 6.22 – Diagonal shear failure of fixed-fixed column under scaled EIGM	143
Figure 6.23 - Direct shear failure for fixed-fixed column under scaled EIGM	144
Figure 6.24 - Comparing damage assessment between fixed-fixed column and Pan et al (2004b)	146
Figure 7.1 - Elevation and details of a 6-Storey RC frame	151
Figure 7.2 - Mode shapes for 6-storey RC frame	152
Figure 7.3- Snapshots at the maximum roof displacement of 229 mm	153
Figure 7.4 - Lateral displacement time histories at different storeys	154
Figure 7.5 - Acceleration time histories at different storeys	155
Figure 7.6 - Inter-storey drift time histories at different storeys	156
Figure 7.7 – Time histories of column damage index at the different storeys	157
Figure 7.8 – Time histories of shear stress at ground storey columns	159
Figure 7.9 – Time histories of normalized bending moment at ground storey columns	160
Figure 8.1 - Phase I shear failure data and extrapolation	164
Figure 8.2 - Phase II global damage indices and damage levels	166
Figure 8.3 - Phase II Shear Failure	167
Figure 8.4 - Phase II column yielding and failure	168

Figure 8.5 - Joint failure for ST2 and ST6 in Phase I and Phase II	170
Figure 8.6 - Damage characteristics from Phase II response analysis	172
Figure 8.7 - Damage characteristics of 6-storey RC frame due to scaled EIGMs	175
Figure 8.8 - Tripartite plot of damage characteristics of a 6-storey frame due to EIGMs	176
Figure 8.9 – Comparison of lower bound of failure in the 2-storey and 6-storey RC frames	177
Figure 8.10 - Postulated relationship between failure of RC frame and height of RC frame	178
Figure 9.1 - Illustration of framework for damage assessment	191

LIST OF SYMBOLS

Chapter 3

a, b	= experimental constants for Malvern's classical equation
A	= uniform cross-sectional area of beam
A_{vf}	= area of shear friction reinforcement per unit width (N/mm ²)
A_{vf}	= area of shear friction reinforcement per unit width (mm ² /mm)
c_1	= longitudinal wave speed, $\sqrt{E/\rho}$
c_2	= shear wave speed, $\sqrt{kG/\rho}$
d	= distance from the extreme compression fiber to the centroid of the tension reinforcement (mm)
DIF	= dynamic increase factor
E	= modulus of elasticity
f_{co}	= 10 MPa
f_{cs}	= static compressive strength
f'_{dc}	= concrete dynamic ultimate compressive stress
f_{dy}	= reinforcement dynamic yield stress
f_y	= static yield strength of steel
$f(\epsilon)$	= quasi-static strength
G	= shear modulus
H	= thickness of the beam
I	= moment of inertia
k	= shear correction factor
$m(x,t)$	= distributed moment
M	= induced bending moment
$M(x,t)$	= induced moment
$M_E(x)$	= the induced moment for EBT
$M_T(x)$	= the induced moment for TBT
N_c	= applied axial compressive force per unit width (N/mm)
$q(x)$	= the distribution of transverse load
$q(x,t)$	= distributed transverse moment

-
- Q = induced shear force
 $Q(x,t)$ = induced shear force
 $Q_E(x)$ = the induced shear for EBT
 $Q_T(x)$ = the induced shear for TBT
 t_d = duration of the IPL
 T = fundamental period of the structure
 $u(z,x)$ = the longitudinal deformation of the beam
 $w(x)$ = the transverse deflection
 x = the distance along the beam
 z = the distance from the neutral axis
 α = $1/(5 + 9f_{cs}/f_{co})$ in Equation (3.8)
 α = dimensionless constant in Equation (3.12)
 α_{fy} = dimensionless constant in Equation (3.12)
 α_{fu} = dimensionless constant in Equation (3.12)
 $\log \beta$ = $7.11\delta - 2.33$ in Equation (3.10)
 $\log \beta$ = $6\delta - 2$ in Equation (3.11)
 $\log \gamma$ = $6.156\alpha - 2$ in Equation (3.8)
 δ = $(10 + 6f_{cs}/f_{co})^{-1}$ in Equation (3.10)
 δ = $(10 + 8f_{cs}/f_{co})^{-1}$ in Equation (3.11)
 ε = strain
 $\dot{\varepsilon}$ = strain rate
 $\dot{\varepsilon}_s$ = static strain rate
 $\phi(x)$ = the rotation of the cross-section
 ρ = density
 σ = strength (in stress)

Chapter 4

- a = for a cantilever column, it is the member length for a column in reversed bending, it is half the member length, in Equation (4.1)
 A_g = gross cross-sectional area of member

A_v	= the total transverse reinforcement
b	= deformation coefficient ratio (taken as 0.77 for RC structures) in Equation (4.9)
c	= depth of the compressive zone of the section, in Equation (4.1)
C	= a strength index taken as a ratio of storey shear to total weight of building above, in Equation (4.6)
C	= damage acceleration coefficient (taken as 1.77) in Equation (4.9)
d	= the column depth
D'	= distance between the centers of the peripheral hoop or spiral
dE	= incremental hysteretic energy
d_F	= roof displacement at the failure of the structure in Equation (4.8)
d_R	= maximum roof displacement in Equation (4.8)
d_Y	= roof displacement at the yield of the first member in Equation (4.8)
E_o	= basic seismic index in Equation (4.6)
f'_c	= concrete compressive strength
f_y	= the steel yield strength
f_{yh}	= the yield strength of the transverse steel reinforcement
F	= a function of the ductility index in Equation (4.6)
G	= geographical index to account for local ground conditions (taken as 1), in Equation (4.6)
GDP	= global damage parameter in Equation (4.8)
h	= overall depth of section, in Equation (4.1)
i	= storey level in Equation (4.6)
I_s	= seismic index in Equation (4.6)
$\frac{l}{d}$	= shear span ratio in Equation (4.10)
M_m^+	= ultimate moment in i -direction in Equation (4.7)
M_x^+	= yield moment in i -direction in Equation (4.7)
M_y^+	= yield moment in i -direction in Equation (4.7)
$MFDR$	= modified flexural damage ratio in Equation (4.7)
n	= number of degrees of freedom in Equation (4.6)
n	= number of structural members in Equation (4.9)

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n_0	= normalized axial force in Equation (4.10)
p	= axial load in Equation (4.1)
P_t	= longitudinal steel ratio in Equation (4.10)
P_y	= yield strength in Equation (4.5)
Q_y	= calculated yield strength in Equation (4.10)
r	= relative plastic strain ratio (i.e. $\frac{\Delta\delta_p^+}{\Delta\delta_p^-}$) in Equation (4.9)
s	= spacing along the longitudinal axis of the member
S_D	= structural design index to account for the level of symmetry in the building and the complexity of structural layout (ranges from 1.2 to 0.4), in Equation (4.6)
T	= time index to account for the loss of strength and age of the Building (ranges from 1.0 to 0.5), in Equation (4.6)
T_1, T_2	= the longitudinal rebar axial forces in beams
V_c	= concrete contribution for diagonal shear strength in Equation (4.1)
V_{col}'	= the shear face on column end
V_{jh}	= the horizontal joint shear force
V_p	= axial load contribution for diagonal shear strength in Equation (4.1)
V_s	= truss-mechanism component for diagonal shear strength in Equation (4.1)
α	= stiffness degrading parameter (Park and Ang, 1985)
β	= strength deterioration parameter (Park and Ang, 1985)
δ_m	= maximum deformation under ground motion in Equation (4.5)
δ_{max}	= maximum deformation experienced at time t in Equation
δ_u	= ultimate deformation during monotonic loading when concrete reaches ultimate strain, in Equation (4.10)
δ_u	= ultimate deformation during monotonic loading when concrete reaches ultimate strain, in Equation (4.10)
δ_u	= ultimate deformation under monotonic loading in Equation (4.5)
$\Delta\delta_f$	= maximum change in plastic deformation to cause failure in Equation (4.9)
$\Delta\delta_p^+$	= incremental positive plastic deformation during i^{th} cycle in Equation (4.9)
ϕ	= storey index in Equation (4.6)

-
- ϕ_m^i = ultimate curvature in i -direction in Equation (4.7)
 ϕ_x^i = arbitrary curvature in i -direction in Equation (4.7)
 ϕ_y^i = yield curvature in i -direction in Equation (4.7)
 γ = pinching parameter (Park and Ang, 1985)
 μ = ductility in Equation (4.6)
 ρ_w = confinement ratio in Equation (4.10)

Chapter 5

- PGA = peak ground acceleration
 PPV = peak particle velocity
 t = time
 t_d = the duration of a load
 y/h = a location along the height of the column

Chapter 6

- a = for a cantilever column, it is the member length for a column in reversed bending, it is half the member length, in Equation (6.16)
 A_g = gross cross-sectional area of member in Equation (6.16)
 A_g = PGA of EIGM
 A_v = the total transverse reinforcement in Equation (6.16)
 A_{vf} = area of shear friction reinforcement per unit width (mm^2/mm) in Equation (6.17)
 c = depth of the compressive zone of the section in Equation (6.16)
 c = the damping of the SDOF system
 C_1 = correction factor for velocity
 C_2 = correction factor for displacement
 C_{2n} = correction factor for displacement of the n^{th} mode of an MDOF system
 d = distance from the extreme compression fiber to the centroid of the tension reinforcement (mm)
 D' = distance between the centers of the peripheral hoop or spiral in Equation (6.16)
 DIF = dynamic increase factor

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f_{yh}	= yield strength of transverse steel reinforcement in Equation (6.16)
f'_{dc}	= concrete dynamic compressive strength in Equations (6.16) and (6.17)
f'_{dy}	= dynamic yield stress of reinforcement in Equation (6.17)
h	= overall depth of section
H	= thickness of the section in the direction of the ground motion in Equation (6.18)
k	= the lateral stiffness of the SDOF system
m	= the mass of the SDOF system
$\mathbf{m}$	= mass matrix for the MDOF system
m_a	= PGA multiplier for scaling EIGMs
m_f	= frequency multiplier for scaling EIGMs
m_v	= PPV multiplier for scaling EIGMs
M	= induced bending moment
M_c	= currently taken as the yielding moment capacity in Equation (6.19)
M_n	= modal mass for the n^{th} mode
MDOF	= multiple degree of freedom system
$\frac{M}{M_c}$	= flexural damage index
N_c	= applied axial compression force per unit width (N/mm) in Equation (6.17)
Q	= induced shear force
Q_c	= shear strength capacity in Equation (6.19)
$\frac{Q}{Q_c}$	= shear damage index
s	= spacing along the longitudinal axis of the member in Equation (6.16)
SDOF	= single degree of freedom system
t	= time
t_d	= time at the end of Phase I
t/t_d	= normalized time
T	= the fundamental period of the structure
T_n	= the fundamental period for the n^{th} mode
$v(t)$	= the relative displacement of the SDOF system at time t

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- v_{du} = direct shear strength capacity in Equation (6.17)
 $v_e(t)$ = approximate relative displacement of the SDOF system at time t
 $v_{ec}(t)$ = the corrected displacement of the SDOF system at time t
 $v_t(t)$ = the absolute displacement of the SDOF system at time t
 $\dot{v}(t)$ = the relative velocity of the SDOF system at time t
 $\dot{v}_e(t)$ = approximate relative velocity of the SDOF system at time t
 $\dot{v}_{ec}(t)$ = the corrected velocity of the SDOF system at time t
 $\ddot{v}(t)$ = the relative acceleration of the SDOF system at time t
 $\ddot{v}_e(t)$ = approximate relative acceleration of the SDOF system at time t
 $\ddot{v}_g(t)$ = ground acceleration applied to the SDOF system at time t
 $\ddot{v}_t(t)$ = the absolute acceleration of the SDOF system at time t
 V = diagonal shear strength capacity in Equation (6.16)
 V_c = concrete contribution in Equation (6.16)
 V_p = axial load contribution in Equation (6.16)
 V_s = truss-mechanism contribution in Equation (6.16)
 $Y_n(t_d)$ = modal response for the n^{th} mode at time $t = t_d$
 $Y_{ne}(t_d)$ = estimated modal responses for the n^{th} mode at time $t = t_d$
 α = $1 - \mu$
 μ = the ratio of time where the apex of triangle is to t_d
 ω = angular frequency of the SDOF system
 θ = induced rotation
 Φ_n = mode shape for the n^{th} mode
 τ = time from the onset of forced vibration
 Γ_n = modal participation factor

Chapter 7

- $|v|$ = the absolute of the induced shear stress normalized by Priestley's shear strength
 $|M/M_u|$ = the absolute of the induced bending moment normalized by the ultimate bending moment capacity

Chapter 8

JF = joint failure

SF = shear failure

ST = storey level

FF = flexural failure

1 INTRODUCTION

1.1 BACKGROUND

A roof over man's heads and that over man's possessions should serve as protection against the elements of nature and those from man's activities. Construction material has over time changed from mud to the more contemporary ones like steel and reinforced concrete. Adoption of any construction material does not change the true essence of what the material serves to provide – a safe haven against the many hazardous elements.

In many countries, reinforced concrete is widely used owing to its fire-resistance and the abundance of its raw materials. In the design of a reinforced concrete (RC) structure, the design philosophy considers the loads and the performance of the structure under these loads, during the structure's lifespan. In some cases, the design loads would include inertial loadings, including ground motions like design earthquakes and even explosion-induced ground motions (EIGMs). In cases of inertial loadings, the performance review would then include some form of damage assessment of the structure induced by these predominantly larger forces. The magnitude of these forces is typically so high that they would have to be resisted past the elastic range of the RC structure (i.e. post-elastic behavior). Owing to the nature of the post-elastic behavior induced by the ground motions, the need for analytical methods to describe the post-elastic behavior prompted their developments.

In the 1920s and 1930s, greater concern was centered around the damaging potential of inertial loads on structures, with seismic design provisions included in design codes. In this respect, the ability of the structure to resist the inertial load would depend on its ability to be subjected to the damaging potential of the inertial load well into its inelastic range. Thus, structural performance can be a measure of the ductility feature of the structure. This is imperative as structures should not experience a brittle failure that is abrupt and allows little time for any evacuation.

Chapter 1: Introduction

Damage assessment serves to be an evaluation of a structural design or as an investigation following a ground motion event. From an engineer's standpoint, damage assessment can shed light to the structural performance, highlighting the damaged regions requiring attention and rehabilitation. From the standpoint of owners, administrators and insurance-providers, such qualitative indications would assist in risk evaluation and so decision-making. This has become a strong motivating factor propelling the advancement in damage assessment techniques.

In the past, earthquake design philosophy involved the design of a structure for a pseudo-dynamic load, representative of the design earthquake. While this offers simplicity, it significantly departs from reality leading to the growing popularity of the capacity design philosophy. Along with it emerged the energy school of thought where the potential of an earthquake (i.e. its energy dissipated to the structures or demand on a structure) was to be transmitted to the structure; the structure absorbs this as potential energy and changes part or all of it to kinetic energy as it sways. Failure would then result from the structure not being able to absorb the energy (i.e. demand exceeding capacity). The advancement in structural strength prediction and structural post-elastic behavior bridged the gap between the actual and the predicted allowing for greater accuracy in section design. In view of the growing popularity of the energy principle, damage assessment based on energy has also been developed on top of the previous damage assessment techniques.

EIGMs being one of the possible damaging elements that may be faced in the lifetime of a structure have been given more attention owing to accidental or even intentional explosions. When EIGMs become the ground motion, adoption of current design philosophy and damage assessment from its smaller magnitude ground motion counterpart of earthquakes seem possible. However, both ground motions (i.e. earthquakes and EIGMs) being inherently different, any direct export of earthquake principles to EIGM cases remains questionable.

Earlier, damage assessment of RC structures subjected to EIGMs has been performed. However, material-level modeling was used which requires significantly

large computational effort. Furthermore, the damage assessment does not clearly reflect the possible failure mechanisms and so decision making is more difficult, in terms of retrofitting and revision of section design.

1.2 PURPOSE AND SCOPE

In this thesis, the research area reported is on RC structures that are subjected to EIGM loads. This section outlines the purpose and scope of the research work. The research work focuses on two areas which are response analysis and damage assessment.

1.2.1 RESPONSE ANALYSIS

For structures subjected to EIGMs, the purpose related to the response analysis is as follows:

1. For designing structures subjected to EIGMs, the design forces needs to be determined. These design forces can be determined by the response analysis of structures subjected to EIGMs. For structural-level modeling, beam theories are used to model the structural members. Current beam theories include Euler-Bernoulli beam theory and Timoshenko beam theory. Timoshenko beam theory has an improved definition of shear deformation and rotatory inertia as compared to the Euler-Bernoulli beam theory. Often, beam theories are represented in the form of finite element models (FEM). For structures subjected to EIGMs, the effect of using different FEM beam theories on the design forces needs to be investigated.
2. When a structure is subjected to EIGMs, the local mode response is significant during the loading duration. As such, the local modes need to be included when modeling a structure, which is subjected to EIGMs. By including the local mode, the number of degrees of freedom in the structural model will increase. This will lead to an increased computational effort as compared to a structural model without the local modes. However, the local mode response of the structure is more significant during the loading duration of the EIGM as compared to the free vibration in the ensuing

Chapter 1: Introduction

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duration. For the response during the free vibration, it is necessary to include the local modes in the structural model. To reduce computational effort, the response analyses during the loading duration and during the free vibration duration should be separated. This will facilitate the inclusion of local modes for the response analysis during the loading duration, while excluding these modes for response analysis during the free vibration. As such, a method that can approximate the response during the loading duration needs to be developed. Also, a method that will facilitate the separation of the response analysis for the loading duration and the free vibration duration. Together, this will reduce the computational effort of the response analysis of structures subjected to EIGMs.

In view of the purpose related to response analysis, the scope is as follows:

1. For structures subjected to EIGMs, investigation shall be made for the use of beam theories in the response analysis. *A comparison shall be made for the response results as determined by the Euler-Bernoulli beam theory and the Timoshenko beam theory.* Both these beam theories shall be represented in their respective finite element models. In Timoshenko beam theory, shear waves propagate at finite speeds as compared to the infinite speeds in the Euler-Bernoulli beam theory. As such, Timoshenko beam theory has been widely recommended for modeling structures under impulsive loads. *The applicability of using Euler-Bernoulli beam theory for structures subjected to EIGM load shall be made.* The induced forces as determined by the two beam theories shall be compared because they are used as design forces.
2. *A method that can approximate the response during the loading duration will be proposed.* This method will be derived in such a way that the response analyses during the loading duration and the free vibration can be separated. With this, the computational effort involved with the response analysis of structures subjected to EIGMs can be reduced.

1.2.2 DAMAGE ASSESSMENT

For the stated research area, the research objectives related to damage assessment are described as follows:

1. The damage assessment for a RC frame is to reflect the failure mechanism that would be experienced. This is different from earlier work such as DIN (1983) and NATO (1977). DIN and NATO have provided regulatory guidelines in terms of peak particle velocity (PPV) and the principal frequency of the ground motion. Being guidelines, they do not reflect the failure mechanism on the structure if any. Thus, a damage assessment that reflects the failure mechanism of a structure subjected to EIGMs needs to be developed.
2. When subjected to an EIGM, structures may experience high strain rates. There is an enhancement of strength due to strain rate effects. This strength-enhancement due to strain rate effects needs to be incorporated into the damage assessment. While the incorporation has been made for structures subjected to impulsive pressure loads (Crawford et al, 2001), this has not been undertaken for the case of EIGM.
3. During the free vibration duration, the RC frame may experience beam-column joint failure in addition to other modes of failure. This type of failure mechanism needs to be considered.
4. For a multi-storey RC frame, the damage assessment for response to EIMGs will require a large computational effort. Thus, a simplified method that will facilitate the damage assessment of the RC frame will speed up the process. A simplified method needs to be developed, and the feasibility of its application needs to be studied.

Related to the damage assessment of structures subjected to EIGMs, the scope is as follows:

1. A damage assessment method that reflects the *failure mechanism* on a structure when subjected to EIMGs is proposed. The failure mode of a multi-storey RC frame will be presented which reflects the failure mechanisms that could be induced by an EIGM. With this achieved, it can

Chapter 1: Introduction

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form the basis of design guidelines for structures resistant to EIGMs. With the failure mechanisms understood, a more resistant design can be made by enhancing the member strength against a particular failure mechanism.

2. The *strength-enhancement effect of higher strain rates* shall be included in the damage assessment of RC frames to EIGM.
3. *Beam-column joint failure* shall be incorporated as a possible mode of failure during the *free vibration phase*. Here, beam-column joint failure is defined by an inter-storey drift greater than or equal to 3 % of the storey height (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003).
4. A *simplified method for damage assessment of an RC building shall be proposed*, and its applicability for use shall be tested. This simplification shall be an attempt to perform the damage assessment of a structure during the loading duration, by using a critical column from the multi-storey RC frame.

1.2.3 SCOPE

In view of the research needs related to the research area stated, the scope of the research can be summarized as follows:

- A comparison shall be made for the response results as determined by the Euler-Bernoulli beam theory and the Timoshenko beam theory
- A method that can approximate the response during the loading duration will be proposed.
- A damage assessment method that reflects the possible *failure mechanism* on a structure when subjected to EIMGs is proposed.
- The *strength-enhancement effect of higher strain rates* shall be included in the damage assessment of RC frames to EIGM.
- *Beam-column joint failure* shall be incorporated into the possible modes of failure for the *free-vibration phase*.
- A *simplified method for damage assessment of an RC building shall be proposed*, and its applicability for use shall be tested.

2 EXPLOSION-INDUCED GROUND MOTIONS (EIGMS)

Explosions-induced ground motions (EIGMs) are ground motions generated from *underground explosions*. As opposed to earthquakes,

- EIGMs occur within a much shorter duration (in the order of microseconds) and so have a higher frequency content (Lu et al, 2001; Pan et al, 2001; Ma et al, 2002; Pan et al, 2002; Pan et al, 2003a; Pan et al, 2004b)
- EIGMs have ground accelerations of larger amplitudes (in the range of 80 g to 100 g) (Lu et al, 2001; Pan et al, 2001; Ma et al, 2002; Pan et al, 2002; Pan et al, 2003; Pan et al, 2004b)

There has been a growing concern of structures subjected to EIGMs as a result of accidental or deliberate impact or explosions.

As EIGMs are occurring within very short durations, they can be considered as an impulsive load. Another type of impulsive loads is impulsive pressure loadings (IPLs) due to blast. While EIGMs are ground motions due to underground explosions, IPLs are pressure loadings due to blasts which are above ground. Figure 2.1 shows examples of IPLs, which are the gas explosion in Ronan Point (1968) and the Oklahoma City terrorist act (1995). IPLs have raised questions about the design philosophy of buildings, be it in active or less active seismic regions.

Figure 2.1 - Extent of damage due to impulsive pressure loadings

Chapter 2: Explosion-Induced Ground Motions(EIGMs)

An EIGM is in effect a shock wave propagating at about 5,000-10,000 m/s through high explosives generating ground motions that can be higher than most earthquakes. The magnitude of the ground motion may depend on factors such as:

- (a) Weight of explosive,   (b) Size of explosive,
- (c) Type of explosive and   (d) Distance from the detonation point,

For the purpose of determining shock wave magnitude, scaling laws have been proposed based on experimental data (Watson, 2002).

2.1 CURRENT EIGM PROVISIONS

Following some of the mentioned events, revisions were made in the various design codes.

In the United Kingdom, BS8110: Part 1:1997 (BSI, 1997) included new provisions being:

- Key elements to be designed for 34 kN/m^2
- Peripheral ties for buildings above ten stories to be designed for a 60 kN tensile force

In the United States of America, the U.S. Department of Defense Explosive Safety Board (DDESB) safety code on underground ammunition storage limits the maximum particle velocity on building sites to be 60 mm/s in soil, 114 mm/s in soft rock and 230 mm/s in hard rock (DDESB, 1996). Other regulatory guidelines that outline the peak particle velocity are DIN (1983) and NATO (1977).

2.2 RESPONSE PHASES

Structural response to EIGMs can be segmented into two parts: the major shock duration (Phase I) and the ensuing duration (Phase II). The two phases of structural response is displayed in Figure 2.2. It was shown that the high frequency nature of a BIGM led to a large shear force with small deformation during Phase I followed by significant deformation in Phase II (Pan et al, 2002; Dhakal and Pan, 2003). Other

studies have shown that the EIGM produces significant distributed local damages owing to the high frequency content of EIGM (Ma et al, 2002). For EIGMs, one indicator used for damaging potential of the ground motion is the impulse per unit mass of the ground motion (Pan et al, 2002).

Figure 2.2 – Illustration of Phase I and Phase II response

In earthquakes, a significant portion of structural displacement can be realized in Phase I response. However, EIGMs occurring within subseconds would induce less structural displacement in Phase I with most of the displacement attained in Phase II. Furthermore, one may expect a higher structural displacement for EIGMs in Phase II, owing to their shorter duration (i.e. higher principal frequency).

2.3 IMPLICATIONS

To summarize, these can be said about structures responding to ground motions which include earthquakes and EIGMs. In general, ground motions can be

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Chapter 2: Explosion-Induced Ground Motions(EIGMs)

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categorized as two types which are those that occur within a very short duration and those occur within a longer duration. When a structure is subjected to a longer-duration ground motion, the structural displacement would be larger during the loading duration (i.e. Phase I). However, when a structure is subjected to a short-duration load, the structure would experience small displacements during Phase I and large induced shear forces. As the structure begins its free-vibration during Phase II, the structural displacement becomes significantly large.

Thus, an RC structure designed for resistance to EIGM, which are of short duration, should have the following considerations:

- The structure should possess *toughness* for the large induced shear forces during Phase I.
- The structure should possess *ductility* for Phase II response, where the displacements are significantly larger.
- In times of chronology, the toughness consideration is prior to the ductility consideration.

The current regulatory guidelines do not relate the possible damage mechanisms that a structure would experience when subjected to EIGMs (DDESB, 1996; DIN, 1983; NATO, 1977). This makes it difficult for the application in engineering design application. Currently, the regulatory guidelines do not take into account how different structures would respond to the EIGMs in different ways. In this case, the regulatory guidelines may appear to be too conservative for some structures. Thus, an assessment of the possible failure mechanisms for RC structures subjected to EIGMs should possess the following characteristics:

- Reflect the failure mechanisms that would be experienced such that section designs can be changed or retrofitting can be undertaken to prevent those failure mechanisms.
- Reflect the failure mechanisms in terms of characteristics of *both* structure and EIGM. As for the characteristics of the EIGM, using the PPV and principal frequency of the EIGM is suitable. In this manner, the design engineer would only have to consider an EIGM with those characteristics

Chapter 2: Explosion-Induced Ground Motions(EIGMs)

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acting on the super-structure. In this case, the EIGM would not have to consider a list of other characteristics such as: (a) distance of the structure from the underground explosion, (b) the depth of underground explosion, (c) the weight of the explosives used and (d) any attenuation of the ground motion as it propagates to the structure from the source of explosion.

3 LITERATURE REVIEW - PHASE I OF EIGM

As mentioned, the structural displacement induced by an EIGM during Phase I is small. Due to the high frequency of the EIGM, the local higher-order modes of the structure show significant response during Phase I. At the same time, the induced shear is significantly large during Phase I. This chapter is a literature review related to concerns that are expected to be important to Phase I such as:

- General EIGM observations,
- Response approximation,
- Structural damping,
- Finite element analysis,
- Strain rate effect and direct shear capacity and
- Strength enhancement due to strain rate effects

3.1 GENERAL EIGM OBSERVATIONS

The following is a summary of earlier observations on EIGMs:

- When structures are subjected to EIGM, the structure shows small displacements with significantly large induced shear forces (Lu et al, 2001; Pan et al, 2001; Ma et al, 2002; Pan et al, 2002; Pan et al, 2003a; Pan et al, 2004b).
- Frames have global mode(s) associated with the floor(s) as main mass points. There are also local modes associated with the response degrees-of-freedom modes at intermediate points along the distributed mass of beams, columns and slabs. When frames are subjected to high-frequency ground motions, the local modes display significant response (Lu et al, 2001; Pan et al, 2002; Dhakal and Pan, 2003). Thus, in the response analysis of frames subjected to EIGMs, inclusion of the higher-frequency local modes within the analytical model is necessary.
- With increasing distance from the point of detonation, the amplitude and frequency of the EIGM decreases dramatically. Also, fractures and joints in the rock mass do filter out the stress wave components (Hao et al, 2001).

- In terms of structural dynamics, stability of the numerical solution requires a sufficiently small time-step during the time-series analysis. For instance, when using the Newmark's method for linear systems, a relationship exists between the time-step used for the ground motion and the highest-order mode within the structure (Chopra, 1995a). Thus, in order to include local mode-participation, the time-step of the EIGM has to be small enough to ensure stability in the numerical solution. Thus, for the response analysis of structures subjected to EIGMs, there is a large computational effort required because of the small time-steps used for the analysis and the large degrees of freedom required for the inclusion of local modes.
- For a two-storey RC frame subjected to an EIGM, material-based modeling showed that the damage along the first-storey column was significantly larger than that of the second-storey column (Ma et al, 2002; Pan et al, 2004b). For a six-storey RC frame subjected to an EIGM, structural-level modeling revealed that the induced shear force at the base of the first-storey column is the largest (Pan et al, 2004b).
- For the two-storey RC frame used by Lu et al (2001), it was noted that second-storey mass *filtered* significant amounts of the high-frequency input energy as the stress waves propagate along the column. The observation of the filtering of higher-frequency input energy is consistent with that made by Boyle (1997). Boyle observed that the presence of concentrated masses can act as frequency *filters* by blocking high-frequency components of waves. This implies that the amount of *filtering* of ground input energy is dependent on the amount of mass.

3.2 APPROXIMATE RESPONSE ANALYSIS

For an IPL idealized as a triangular pulse, there are three regimes of response:

- Impulsive,
- Quasi-static and
- Dynamic (Smith and Hetherington 1994).

Classification of the different regimes was made based on the two variables:

Chapter 3: Literature Review – Phase I of EIGM

- t_d = the duration of the IPL
- T = the fundamental period of the structure.

The different regimes depended on the ratio t_d/T . For the triangular load considered, Smith and Hetherington (1994) stated that it is an impulsive load if t_d/T is less than $0.2/\pi$.

Also adopting the t_d/T ratio, Clough and Penzien (1993) stated that for a similar triangular load and other loads such as rectangular and half-sine wave, these loads can be considered as a short-duration load if t_d/T is less than 0.25. For a short duration load, Clough and Penzien (1993) concluded that the maximum displacement depends upon the magnitude of the applied impulse, where the magnitude of the applied impulse is the integration of the load with respect to time. In addition, they also showed an approximate method for determining the response of a structure subjected to a short-duration load, by using the impulse-momentum relationship. Given the short loading duration inherent of EIGMs, the mentioned impulse-momentum relationship can be used to determine the Phase I response. As it is an approximate method, the level of computational effort would be less than the material level modeling.

A closed-form analytical solution can be developed for specific shapes of short-duration load. Thus, approximate response such as displacement, velocity and induced forces can be expressed as a function of time. Further, the velocity and displacement at the end of impulse when time $t = t_d$ can be used as the initial conditions for response analysis of Phase II. As Phase II response is predominantly global responses, by using the approximate response at time $t = t_d$ as the initial conditions, Phase I and II response analysis can be segregated. This means that the response analyses of Phase II will not be constrained by the large number of degrees of freedom used and extremely small time-steps required for Phase I analysis. This leads to less computational effort and thus more efficient response calculation for Phase II.

3.3 STRUCTURAL DAMPING

In structural dynamics, the dynamic equilibrium equation by d’Alambert’s principle includes a force-term for the damping effects. In earthquake engineering, the damping force is almost always included in the response analysis. Considerations have been made as for the inclusion of the damping force in the response analysis for structures subjected to impulsive loads.

In the study by Kiger and Salim (1998), a structure was subjected to an impulsive pressure loading. The response analysis was performed by including structural damping and later repeated without including structural damping. It was observed that structural displacement decreased with increasing damping. In cases of impulsive loads where structural displacements are significantly small, excluding structural damping would produce larger induced forces as compared to including structural damping. Thus, for the design of the sections, a calculated induced force that is larger would be considered as conservative. For the same reason, other response analysis of structures subjected to impulsive pressure loadings also excludes the structural damping (Crawford et al, 1994).

For structures subjected to EIGM within this thesis, the structural damping shall therefore be excluded for the response analysis during Phase I.

3.4 BEAM THEORIES

There are several beam-column finite elements available. For determining the structural response induced by EIGMs, the choice of a suitable beam theory needs to be made. The available beam theories include:

- a) Euler-Bernoulli beam theory (EBT) – First-order deformation
- b) Timoshenko beam theory (TBT) – Second-order deformation
- c) Reddy-Bickford beam theory (RBT) – Third-order deformation

Figure 3.1 is a summary of the deformation characteristics of the above beam theories (Wang et al, 2000).

Here, RBT shall not be discussed. Notably, there is an observation made by Wang et al (2000) regarding the higher-order beam theories, which is:

“In higher-order theories, the EBT is further relaxed by removing the straightness assumption. Theories higher than higher-order are seldom used because the accuracy gained is so little that the effort required to solve the equations is unjustified.”

Figure 3.1 - Deformation for three beam theories (Wang et al, 2000)

3.4.1 EULER-BERNOULLI BEAM THEORY (EBT)

The EBT is a widely used beam-column element. It is assumed that straight lines normal to the mid-plane of the beam remain straight and normal to the mid-plane after deformation. For a prismatic and homogeneous bar, the constitutive and equilibrium equations of EBT are shown in Equations 3.1 and 3.2.

Chapter 3: Literature Review – Phase I of EIGM

Constitutive Equation

The displacement field for EBT is shown in Equation 3.1 with the force equilibrium shown in Equation 3.2.

$$u(z, x) = -z \frac{dw(x)}{dx} \quad (3.1)$$

where

z	=	the distance from the neutral axis
x	=	the distance along the beam
$u(z, x)$	=	the longitudinal deformation of the beam
$w(x)$	=	the transverse deflection

Equilibrium Equation

$$M^E(x) = -EI \frac{d^2 w}{dx^2} \quad (3.2a)$$

$$Q^E(x) = -EI \frac{dw}{dx} \quad (3.2b)$$

$$q(x) = -\frac{d^2 M^E(x)}{dx^2} \quad (3.2c)$$

$$q(x) = -\frac{dQ^E(x)}{dx} \quad (3.2d)$$

where

$M^E(x)$	=	the induced moment for EBT
$Q^E(x)$	=	the induced shear for EBT
E	=	modulus of elasticity
I	=	moment of inertia
$q(x)$	=	the distribution of transverse load

3.4.2 TIMOSHENKO BEAM THEORY (TBT)

The TBT is a second-order beam theory as compared to the first-order EBT. On the other hand, the TBT has relaxed the straightness assumption in EBT. In addition, a *constant state of transverse shear strain* γ with respect to the depth of the beam is

Chapter 3: Literature Review – Phase I of EIGM

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included. For a prismatic and homogenous bar, the constitutive and equilibrium equations of TBT are shown in Equations 3.3 and 3.4.

Constitutive Equation

Notice that there is an additional displacement field γ_{xz} .

$$u(z, x) = -z\phi \quad (3.3a)$$

$$\gamma = \frac{\partial u(z, x)}{\partial z} + \frac{\partial w(x)}{\partial x} = \phi + \frac{dw(x)}{dx} \quad (3.3b)$$

where

- z = the distance from the neutral axis
- x = the distance along the beam
- $u(z, x)$ = the longitudinal elongation of the beam
- $\phi(x)$ = the rotation of the cross-section
- $w(x)$ = the transverse deflection for TBT

Equilibrium Equation

$$M^T(x) = EI \frac{d\phi(x)}{dx} \quad (3.4a)$$

$$Q^T(x) = kGA \left(\frac{dw(x)}{dx} + \phi(x) \right) \quad (3.4b)$$

$$q(x) = -\frac{d^2 M^T}{dx^2} = -\frac{d^2}{dx^2} \left(EI \frac{d\phi}{dx} \right) \quad (3.4c)$$

$$q(x) = -\frac{dQ^T}{dx} = \frac{d}{dx} k \left(GA \frac{dw(x)}{dx} + GA\phi(x) \right) \quad (3.4d)$$

where

- $M^T(x)$ = induced moment for TBT
- $Q^T(x)$ = induced shear for TBT
- E = Young's modulus
- G = shear modulus
- A = cross-sectional area of beam
- k = shear correction factor

3.4.3 DYNAMIC ANALYSIS - TBT vs. EBT

This section compares the TBT with EBT in dynamic analysis. Here, a prismatic and homogenous beam is used for the discussion.

Equilibrium Equation (for both TBT and EBT)

Force-equilibrium:

$$\frac{\partial}{\partial x} Q(x, t) + q(x, t) = \rho A \frac{\partial^2}{\partial t^2} w(x, t) \quad (3.5a)$$

Moment-equilibrium:

$$\frac{\partial}{\partial x} M(x, t) - Q(x, t) + m(x, t) = \rho I \frac{\partial^2}{\partial t^2} \phi(x, t) \quad (3.5b)$$

where

ρ	=	density
A	=	uniform cross-sectional area
I	=	area moment of inertia
$Q(x, t)$	=	induced shear
$M(x, t)$	=	induced moment
$m(x, t)$	=	distributed moment
$q(x, t)$	=	distributed transverse load

Unique only to TBT and not EBT is that the substitution of Equations 3.4a and 3.4b into Equations 3.5a and 3.5b leads to two coupled second-order differential equations. When the two coupled equations are transformed into a single forth-order differential equation, Equation 3.6 arises.

$$\begin{aligned} & \left(\frac{\partial^2}{\partial t^2} - c_2^2 \frac{\partial^2}{\partial x^2} \right) \left(\frac{\partial^2}{\partial t^2} - c_1^2 \frac{\partial^2}{\partial x^2} \right) w(x, t) + \frac{A}{I} c_2^2 \frac{\partial^2 w(x, t)}{\partial t^2} \\ &= \frac{c_2^2}{\rho I} \left(q + \frac{\partial m}{\partial x} \right) - \frac{1}{\rho A} \left(c_1^2 \frac{\partial^2 q(x, t)}{\partial x^2} - \frac{\partial^2 q(x, t)}{\partial t^2} \right) \end{aligned} \quad (3.6)$$

where

c_1	=	longitudinal wave speed, $\sqrt{E/\rho}$
c_2	=	shear wave speed, $\sqrt{kG/\rho}$

Table 3.1 shows the longitudinal wave speeds and shear wave speeds for reinforced concrete and aluminium.

Table 3.1 - Shear and longitudinal wave speeds for stated materials

	Reinforced Concrete	Aluminium (6061-T6)
E (GPa)	25.92	70.00
G (GPa)	10.80	26.00
ρ (kg/m ³)	2,450	2,700
c_1 (km/s)	3.3	5.0
c_2 (km/s)	2.1	3.4

Implication

As Equation 3.6 is only evident in TBT and not EBT, the wave speeds that are introduced, which are material-dependent, is only evident in TBT. These wave speeds describe the speeds that longitudinal and shear waves propagate along a Timoshenko beam continuum. On the other hand, the waves propagate at infinite speeds for Euler-Bernoulli beam continuum. Such an observation has been widely made by many researchers including Antes et al (2004) and Au et al (2003).

3.4.4 FINITE ELEMENT METHOD (FEM)

With increasing computer processing speeds, there has been greater use of the finite element method (FEM) in providing solutions to the differential equations within structural mechanics and structural dynamics. However, FEM offers solutions to response analysis by discretizing a continuum structure. Documentation of the FEM for TBT and EBT can be found in Cheng (2000).

FEM: TBT and EBT

As compared to the EBT-FEM, the TBT-FEM introduces a significantly larger computational effort. As the TBT takes into account the shear and rotatory inertia, a

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Chapter 3: Literature Review – Phase I of EIGM

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consistent mass matrix needs to be generated when using the TBT-FEM. The consistent mass matrix of TBT-FEM as compared to the lumped mass matrix for the EBT-FEM is larger and thus increases the computational effort. On the other hand, can EBT-FEM with sufficient FEM refinement (i.e. using more elements) provide sufficiently accurate results?

Antes et al, 2004

In a study by Antes et al (2004), a comparison was made among the following cases:

- a) FEM based on EBT (EBT-FEM),
- b) FEM based on TBT (TBT-FEM) and
- c) Boundary element method

For FEM, the number of elements used was 4, 8 and 20. The numerical investigation was performed using a fixed-simply supported steel beam subjected to a harmonic lateral point load at the mid-span of the beam. The first three modes of beam had modal frequencies of about 350 Hz, 1000 Hz and 1300 Hz. The principal frequency of the lateral point load used was 72,000 Hz. In short, the lateral point load used had a principal frequency between about 200 times of the first structural modal frequency, 72 times of the second structural modal frequency and 50 times of the third structural modal frequency. Figure 3.2 shows the plot of beam end rotation against frequency for the three categories (Antes et al, 2004).

The following observations/implications can be made:

- a) When using the same number of elements for TBT-FEM and EBT-FEM, the modal frequency for TBT-FEM is lower than EBT-FEM. In short, TBT produces a “softer” structure than EBT.
- b) For the same number of elements, the beam end rotation as determined by TBT-FEM and EBT-FEM matched up to almost the third mode. Up to this mode, the principal frequency of the load is about 50 times of the modal frequency of this mode. When the principal frequency of the load is more than about 50 times of the modal frequency of the mode, the beam end

rotation is significantly different. However, the effect of the difference in beam end rotation on induced shear forces and bending moment needs to be determined.

- c) This implies that each modal response as determined by TBT and EBT is similar when the frequency of the mode is less than 1/50 times of the principal frequency of the load.
- d) Although the above is observed, it should be noted that the modal participation factor of high-order modes are significantly lower than lower-order modes. In the case of a given beam, deformations are determined by the modal superposition of the various modes. In a case where there is a significant difference in a higher-order modal response between EBT-FEM and TBT-FEM, this difference when combined by modal superposition to determine the beam deformation may be no longer as significant. This is due to a smaller modal participation factor from these higher-order modes and so a smaller contribution during modal superposition.
- e) In conclusion, while TBT offers a greater accuracy than EBT. It can be said with *sufficient FEM refinement* in EBT, the response analysis of a structure would provide sufficiently accurate results.

FEM vs. Continuum Models

When FEM is used, it treats a continuum system as a system of discrete elements. In reality, the system is in fact a continuum. When structures are subjected to EIGMs, the FEM refinement process should be sufficiently fine such that the results are accurate (Lu et al, 2001). This is due to the high-frequency content of EIGM and so the need to include the higher-frequency, local modes. However, does FEM refinement provide sufficiently accurate results to response analysis? The research by Antes et al (2004) shows that FEM methods do provide sufficiently accurate solutions, as compared to continuum models. In developing a fundamental solution for Timoshenko beams and comparing the results with FEM solutions, the comparison showed that with sufficient FEM refinement, the two solutions do converge.

Chapter 3: Literature Review – Phase I of EIGM

Figure 3.2 - Frequency resonance dependence of beam deformation (Antes et al, 2004)

3.5 STRAIN RATE EFFECT

It has been hypothesized and also demonstrated that strength enhancements in materials occur with increased strain rate. This has been termed the strength-enhancement effect at high strain rates. This strength-enhancement effect at high strain rates can be attributed to the velocity at which a crack propagates through the material. In the case of brittle materials such as concrete, the material failure is dominated by fracture with little or no plastic deformation. In this case, there exists a limited crack velocity for the crack propagation in this fracture failure mode. For the crack to propagate at a higher velocity than the limiting crack velocity, the fracture will not occur as readily but with increased stress and deformation. Such a phenomenon will lead to increased strength at high strain rates (Tedesco et al, 1997). On the other hand, in ductile materials where plastic deformation dominates fracture, much higher strain rates will effect the strength enhancements. The increase in the strength of the material at higher strain rates is normally stated as a factor, which is known as the dynamic increase factor (DIF).

3.5.1 CLASSICAL EQUATION: MALVERN

In the mid-twentieth century, the classical equation relating strength and strain rate was developed by Malvern (1951).

$$\sigma = f(\varepsilon) + a \ln(1 + b\dot{\varepsilon}) \quad (3.7)$$

where

σ	=	strength (in stress)
ε	=	strain
$f(\varepsilon)$	=	quasi-static strength
a, b	=	experimental constants

Following this, much experimental studies have led to the development of relationships describing the strength-enhancement at high strain rates. For both the tensile strength-enhancement and the compressive strength-enhancement in concrete, the Split-Hopkinson pressure bar apparatus was used, with the specimen

orientated differently to produce the direct compressive tests and the splitting tensile tests.

3.5.2 CONCRETE COMPRESSIVE STRENGTH-ENHANCEMENT

There are several proposed DIF for concrete compressive strength-enhancement and two are shown below.

CEB (1993)

Compression strength-enhancement relationships were outlined in the Comité Euro-International du Béton (CEB) Model Code (CEB, 1993). Equation 3.8 below shows the concrete compressive DIF defined by CEB (1993). Figure 3.3 shows the concrete compressive DIF by CEB for the strain rates ranging from $1\text{E-}5\text{ s}^{-1}$ to 100 s^{-1} . From Figure 3.3, it can be seen that there is an abrupt change in slope of the DIF for concrete compressive strength at a strain rate of 30 s^{-1} .

$$\text{DIF} = \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1.026\alpha} \quad \text{for } \dot{\epsilon} \leq 30\text{ s}^{-1} \quad (3.8a)$$

$$\text{DIF} = \gamma \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1/3} \quad \text{for } \dot{\epsilon} > 30\text{ s}^{-1} \quad (3.8b)$$

where

- $\dot{\epsilon}$ = strain rate in the range of 30×10^{-6} to 300 s^{-1}
- $\dot{\epsilon}_s$ = static strain rate of 30×10^{-6}
- α = $1/(5 + 9f_{cs}/f_{co})$
- f_{cs} = static compressive strength (MPa)
- f_{co} = 10 MPa
- $\log \gamma$ = $6.156\alpha - 2$

Figure 3.3 - Concrete compressive strength-enhancement (CEB, 1993)

Tedesco et al (1997)

Another group of researchers produced the regression analysis on the direct compressive tests for compressive stresses ranging from 28 MPa to 57 MPa (Tedesco et al, 1997).

$$\text{DIF} = 0.758 \log \dot{\epsilon} - 0.289 \leq 2.5 \text{ for } \dot{\epsilon} > 2.32 \text{ s}^{-1} \quad (3.9a)$$

$$\text{DIF} = 0.758 \log \dot{\epsilon} + 1.058 \leq 1.0 \text{ for } \dot{\epsilon} \leq 2.32 \text{ s}^{-1} \quad (3.9b)$$

This concrete compressive strength-enhancement and that developed for concrete tensile strength-enhancement have been incorporated into the concrete constitutive model in a finite-element computer program, ADINA (1987).

3.5.3 CONCRETE TENSILE STRENGTH-ENHANCEMENT

There are several proposed DIF for concrete tensile strength-enhancement. Here, two are featured below.

CEB (1993)

The DIF for concrete tensile strength as developed by CEB (1993) is as follows.

$$\text{DIF} = \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1.016\delta} \quad \text{for } \dot{\epsilon} \leq 30 \text{ s}^{-1} \quad (3.10a)$$

Chapter 3: Literature Review – Phase I of EIGM

$$\text{DIF} = \beta \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1/3} \quad \text{for } \dot{\epsilon} > 30 \text{ s}^{-1} \quad (3.10b)$$

where

$\dot{\epsilon}$ = strain rate in the range of 3E-6 to 300 s⁻¹

$\dot{\epsilon}_s$ = static strain rate of 3E-6 s⁻¹

δ = $1/(10 + 6f_{cs} / f_{co})$

f_{cs} = static concrete compressive strength (MPa)

f_{co} = 10 MPa

$\log \beta = 7.11\delta - 2.33$

Malvar and Ross (1995)

Here, a new formulation was proposed for the concrete tensile DIF with slight variations from the CEB (1993) introducing a different strain rate for the abrupt change in DIF.

$$\text{DIF} = \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{\delta} \quad \text{for } \dot{\epsilon} \leq 1 \text{ s}^{-1} \quad (3.11a)$$

$$\text{DIF} = \beta \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1/3} \quad \text{for } \dot{\epsilon} > 1 \text{ s}^{-1} \quad (3.11b)$$

where

$\dot{\epsilon}$ = strain rate in the range of 1E-6 to 160 s⁻¹

$\dot{\epsilon}_s$ = static strain rate of 3E-6 s⁻¹

$\log \beta$ = $6\delta - 2$

δ = $(10 + 8f_{cs} / f_{co})^{-1}$

f_{cs} = static concrete compressive strength (MPa)

f_{co} = 10 MPa

3.5.4 STEEL REINFORCEMENT STRENGTH-ENHANCEMENT

Malvar and Crawford (1998b) proposed a DIF formulation for the steel reinforcement to include the enhancement on tensile strength due to strain rate effect. Equation 3.12 below shows the formulation:

$$\text{DIF} = (\dot{\epsilon} \times 10^4)^\alpha \quad (3.12a)$$

when at yield stress, $\alpha = \alpha_{fy}$

$$\alpha_{fy} = 0.074 - 0.040 \frac{f_y}{414} \quad (3.12b)$$

and for the ultimate stress, when $\alpha = \alpha_{fu}$

$$\alpha_{fu} = 0.019 - 0.009 \frac{f_y}{414} \quad (3.12c)$$

where

$\dot{\epsilon} =$ strain rate

$f_y =$ static yield stress of steel reinforcement (MPa)

$\alpha, \alpha_{fy}, \alpha_{fu} =$ dimensionless constants

3.6 DIRECT SHEAR

In earthquake engineering, the induced shear force that normally results is the diagonal-tensile force, which is commonly known as diagonal shear. For structures the ratio of the induced shear force to the induced moment is lower compared to the same structure subjected to impulsive transverse loads. The principal stresses due to the combination of the induced shear force and induced moment lies on a plane that is inclined at an angle to the longitudinal axis. This inclined manner that results from the interaction of induced shear force and induced moment is known as diagonal shear.

However, when a structure is subjected to impulsive load, the induced shear force is large. Thus, when structures are subjected to EIGMs, the ratio of the induced shear force to induced moment is much higher than when the same structure is subjected to earthquakes. Here, the principal stresses formed are less inclined and more

parallel to the longitudinal axis of the structural member. This type of shear is known as direct shear.

As such, the diagonal shear capacity and the direct shear capacity would be different for a given section. In terms of engineering designs, it is vital to determine which shear capacity should be used based on which of the two phenomena of direct shear and diagonal shear would be experienced.

3.6.1 SHEAR HINGE

In the research by Li and Jones (2000), a separation was made between the occurrence of direct shear and diagonal shear. Without having to determine the formation of principal stresses and the angle in which principal stresses were formed, their research allowed for one to determine which of the two shear forces would occur based on the ratio of induced shear force to induced moment. This was described in the extent of the formation of a shear hinge. Shear hinge is when there is an occurrence of shear dominant deformation.

For a rectangular cross-section, the extent to the formation of a shear hinge is

$$\frac{M}{Q} \leq 0.433H \quad (3.13)$$

where

M = induced bending moment

Q = induced shear force

H = thickness of the beam

This implies that for the formation of a shear dominant zone and so exhibiting direct shear, Equation 3.14 below would have to be exhibited:

$$\frac{Q}{M} \geq \frac{1}{0.433H} \quad (3.14)$$

3.6.2 DIRECT SHEAR CAPACITY

The direct shear capacity for concrete section was developed by Malvar and Crawford (1998a) based on experimental research. Such a shear capacity allowed for the inclusion of the strength-enhancement of concrete at higher strain rates.

$$v_{du} = 0.14f'_c + \frac{1.2}{d}(N_c + A_{vf}f_{dy}) \leq 0.6f'_{dc} \quad (3.15)$$

where

N_c = applied axial compression force per unit width (N/mm)

A_{vf} = area of shear friction reinforcement per unit width (N/mm²)

d = distance from the extreme compression fiber to the centroid of the tension reinforcement (mm)

f'_{dc} = concrete dynamic ultimate compressive stress (MPa)

f_{dy} = reinforcement dynamic yield stress (MPa)

In Equation 3.15, while f'_{dc} can be determined by Equation 3.8, f_{dy} can be determined by Equation 3.12.

Analysis and Experiment

While testing their retrofitting technique, Crawford et al (2001) performed experimental testing and numerical analysis of a column subjected to impulse pressure loading due to blasts. The detonation used ranged from about 400 kg to 800 kg of TNT and the distance between the column and the detonation ranged from about 3 m to 6 m. Figure 3.4 shows the experimental observations and analytical results of a column that failed when subjected to one of the blast loadings. The direct shear as observed between experimental work and numerical simulation matches well.

3.7 DAMAGE ASSESSMENT

When subjected to an EIGM, a structure experiences small deformations and large induced shear forces during Phase I. With the small deformations, response-based damage assessment techniques would not be suitable because the damage level as

determined would be lower than what is experienced by the structure. Thus, response-based damage assessment techniques widely-used in earthquake engineering is not suitable for Phase I response. The recommendation of not using earthquake engineering damage assessment for structures subjected to EIGMs have been made in previous research (Ma et al, 2002; Pan et al, 2002).

Figure 3.4 - Observation and simulation of a column subjected to blast loading
(Crawford et al, 2001)

3.8 IMPLICATIONS

For the Phase I response, the damage assessment techniques should not be response-based. As the shear is highest at the base of a ground-storey column, damage assessment of a structure subjected to EIGMs for Phase I can be a strength-based damage index based on column shear. However, such a damage index would have to use the direct shear strength capacity, which incorporates the effect of strength-enhancement due to strain rate effects.

As the response analysis during Phase I requires a sufficiently refined FEM model, to proceed both Phase I and Phase II response analysis in a single analysis would require too much computational effort. This is because of the large number of degrees of freedom required during Phase I together with very small time steps.

Chapter 3: Literature Review – Phase I of EIGM

Thus, an approximate method that can approximate the response of the structure at the end of Phase I would save computational effort in Phase II. This is because the Phase II response can then be carried out separately with a larger time stepping and fewer degrees of freedom in the FEM model.

4 LITERATURE REVIEW – PHASE II OF EIGM

Under EIGM, the displacements are negligible in Phase I but may become significantly larger in Phase II. As such, the large deformations indicate the favorable use of earthquake engineering techniques. This includes post-elastic behavior and member strength predictive equations.

The capacity design of RC structures describes the design of structures for earthquakes as the designer taking control of the post-elastic behavior. By this, the designer controls which and where the plastic hinges are to occur but avoids undesirable collapse mechanisms. Procedurally, the capacity design philosophy involves:

1. Defining the plastic hinge regions
2. Detailing of elemental capacity within plastic regions so that their associated strengths are adequate
3. Detailing of the structure to avoid undesirable collapse mechanisms

4.1 MEMBER STRENGTH

Structural elements can be broadly divided into beam and column members and beam-column joints (joints for short). While members resist a combination of axial, shear, flexure and torsion effects, joints serve to transfer the forces within the assembly of members (i.e. subassemblage). Both the conservative and capacity design philosophies stipulate that any element is to be designed for resisting the load through strength adequacy. Earlier experimentation has recommended predictive strength equations where some have been adopted by design codes. In view of EIGMs, a critical region is the column base at ground storey where large shear forces are developed. Here, only column shear strength shall be discussed. This section discusses the diagonal shear capacities.

In any concrete section, the shear strength contribution has been envisioned to comprise the contribution from concrete and reinforcement. While the shear

strength of concrete depends greatly on the concrete compressive strength, the shear strength of the reinforcement directly hinges on the reinforcement and its detailing.

Shear Strength of Priestley et al (1994)

While other diagonal shear strength capacities have been developed such as that by ASCE-ACI Committee 426 (1973), the one by Priestley et al (1994) compares much better with experimental results.

In his formulation, the shear strength comprised:

1. Concrete contribution, V_c
2. Axial load contribution, V_p
3. Truss-Mechanism component, V_s

$$V = V_c + V_p + V_s \quad (4.1a)$$

Concrete Component, V_c

For the section, concrete contribution is proportional to the root of the concrete strength.

$$V_c = 0.29\sqrt{f'_c}(0.8A_g) \quad (4.1b)$$

where

f'_c = concrete compressive strength

A_g = gross cross-sectional area of member

Axial Load Component, V_p

In the formulation by Priestley et al (1994), the axial load component is segregated from the concrete contribution and given as follows:

$$V_p = \frac{h-c}{2a}P \quad (4.1c)$$

where

h = overall depth of section

c = depth of the compressive zone of the section

Chapter 4: Literature Review – Phase II of EIGM

- $a =$ for a cantilever column, it is the member length; for a column in reversed bending, it is half the member length
- $p =$ axial load

Truss-Mechanism Component, V_s

Unlike the ASCE-ACI 426 equation (1993) which uses a 45° diagonal crack, the Priestley formulation adopts a 30° diagonal crack, as given below:

For rectangular columns:

$$V_s = \frac{A_v f_{yh} D'}{s} \cot 30^\circ \quad (4.1d)$$

where

- $A_v =$ the total transverse reinforcement
- $s =$ spacing along the longitudinal axis of the member
- $D' =$ distance between the centers of the peripheral hoop or spiral
- $f_{yh} =$ the yield strength of the transverse steel reinforcement

4.2 JOINT STRENGTH

While member strengths are important considerations, joint strength adequacy is also imperative. It was noted that joints are often the weakest region in a structure by Paulay et al (1978). Supporting observations were made from damaged joints present in severely damaged buildings during the strong earthquakes in Chile of the early 1960s. This prompted greater research efforts in joints, concentrated mainly in New Zealand, Japan and the United States of America where revisions and provisions were made to building codes subsequently.

Joints act to transfer forces between beams and columns and so their state reflects the extent of the load transfer mechanism. When the flow of these forces is interrupted with joint strength exceedance, excessive drift of a structure can be expected which demonstrates the importance of joint performance.

Joint performance has been observed to be highly dependent on joint shear strength (Paulay et al, 1978). During ground motions, while the moments in the columns

Chapter 4: Literature Review – Phase II of EIGM

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above and below the joint are of opposite signs, the moments at the ends of the beams are also of opposite signs (Figure 4.1a). Within a subassembly, the joint experiences horizontal and vertical shear stresses that are normally higher than those developed in the beams and columns (Figure 4.1b and Figure 4.1c). Where high bond stresses are developed, bond deterioration may occur resulting in tensile forces developed in the rebar penetrating through the joint core. As a result, the same rebar on the other side, previously under compression, will now be in tension. Thus, member flexural strength will be compromised, posing a detrimental effect on the overall structural strength.

Paulay and Priestley (1992) have summarized design guidelines for joints which are:

- a. Joint strength should not be less than the shear force associated during any plastic hinge development (in line with the capacity design philosophy)
- b. Column capacity should not be compromised by the joints
- c. Joints should ideally be responding within the elastic range during low ground motions.
- d. Joint detailing should not pose construction difficulty
- e. Bond deterioration should be prevented

With the growing concern of joints, many experimental studies were performed coupled with development in the analysis of the joint shear-resisting mechanisms (joint models).

Figure 4.1 - Statics of an interior beam-column joint

4.2.1 JOINT ANALYSIS

Till date, a number of joint models have been developed where the truss model and the diagonal strut model are popular. The difference between the two is the load resisting mechanism that is theorized, where better understanding can be gained by reviewing the joint forces proposed by Paulay et al (1978).

When the structure drifts in one direction, forces would develop in the joint concrete core and the rebars within. The joint core would experience a state of inclined compression spanning from one joint corner to the opposite corner (diagonal compression) with some tension developed spanning across the two other corners (diagonal tension). Whether the diagonal compression or diagonal tension is higher is not clear but certainly load-dependent. The tension in rebars spanning across the joint would be in opposite states of axial loading - a rebar may be in compression on one side of the joint but in tension on the other side (Figure 4.2). This opposite state of axial load holds true for both beam and column rebars. From equilibrium, shear forces would be developed within the joint in both the horizontal and vertical directions (i.e. V_{jv} and V_{jh} in Figure 4.2). When the structure drifts in the other direction, the rebar axial loads would change but still having opposite states across the joint. By this time, diagonal tensile cracks within the joint concrete core may be present from the pervious drift direction.

Through the analysis of these joint forces, Pualay et al (1978) demonstrated that the horizontal joint shear can be expressed as the forces developed in the rebars and concrete stated in Equation 4.2.

$$V_{jh} = T_1 + T_2 - V_{col}' \quad (4.2)$$

where

$V_{jh} =$	the horizontal joint shear force
$T_1, T_2 =$	the longitudinal rebar axial forces in beams
$V_{col}' =$	the shear face on column end

As seen in Equation 4.2, the joint horizontal shear force formulation does not include any term for bond forces and so can be seen to be unaffected by any bond

deterioration. The difference between the truss model and the diagonal strut model would be the manner of resisting V_{jv} and V_{jh} .

Diagonal Strut Model

The diagonal strut model relates the shear force, V_{jh} , to a diagonal concrete strut within the joint (Figure 4.2) and so depends greatly on the concrete strength. As joint shear forces are developed, there would be the presence of a diagonal strut in compression. The postulation that the diagonal concrete strut being the shear-resisting mechanism implies that any formation of a plastic hinge (i.e. near its face) would impair the development of the diagonal strut mechanism. Supporting experimental observations were made by Paulay et al (1978), Ehsani and Wight (1985), Kurose et al (1991), Joh et al (1991) and Sugano et al (1991).

Truss Model

An alternative to the diagonal strut model is the truss mechanism (Figure 4.2). This model relates the shear to be resisted by two components - a concrete diagonal compression field (not a single strut) and a combination of resistance from the horizontal and vertical reinforcement within the joint core. However, experimental observations have contradicted the truss model's mechanism (Lee et al, 1977; Uzumeri, 1977; Kitayama et al, 1991).

Comparison: Diagonal Strut Model vs. Truss Model

A true joint model remains to be found but should it exist, it would lie between the diagonal strut model and truss model. However, it was noted that a true joint model would not be an arithmetic sum of the diagonal strut model and the truss model (Meinhert and Jirsa, 1981). Research has shown that the diagonal strut model contribution to joint strength outweighs that from the truss mechanism (Leon, 1990; Joh et al, 1991).

Figure 4.2 - Forces in a joint and joint models

4.2.2 JOINT EXPERIMENTATION

From experiments, joint strength was observed to be affected by:

1. Column Axial Load
2. Shear reinforcement
3. Bond slip

Column Axial Load

The effect of column axial load is perhaps one of the most widely studied factors, which was later discovered by many researchers as having an insignificant bearing on joint strength (Hanson and Connor, 1967; Lee et al, 1977; Meinhert and Jirsa, 1981; Kitayama et al, 1991; Fujii and Morita, 1991; Bonaccia and Pantapozoulou, 1993). It was concluded by Kitayama et al (1991) that column axial load had an insignificant effect on joint strength when it was less than half the concrete strength (i.e. $0.5f'_c$).

Shear Reinforcement

Some researchers feel that joint reinforcement can increase joint strength through its load carrying mechanism if not through its confining effect on the joint concrete core (Uzumeri, 1977; Hanson and Connor, 1967). Contradictory observations of the significance of this contribution have also been made (Fujii and Morita, 1991).

Bond Slip

Although not investigated in the 1960s when the onset of joint research was undertaken, more attention was paid to bond slip in the mid-1970s. Several key observations were made, including:

1. Effect on joint failure

Betero and Popov (1977) noted the conversion of tension-controlled failure to compression-controlled failure resulting from bond slip as discussed earlier.

Chapter 4: Literature Review – Phase II of EIGM

2. Recommended anchorage length to reduce bond slip

A recommendation of an anchorage length equivalent to 28 times the rebar diameter, d , was made by Leon (1990)

3. Allowable joint stress

In view of bond slip, researchers recommended an upper limit to the allowable joint stress in the form of $k\sqrt{f_c'}$, where k is a constant, which is tabulated below.

Table 4.1 - Maximum allowable joint stress

Researcher	k
Ehsani and Wight (1985)	0.87
Ehsani et al (1987)	0.63 to 1.07

4. Effect of column depth

With the earlier understanding of the bond failure, some researchers encouraged a larger column depth to alleviate bond failure. While Leon (1990) recommended a column depth of $28d$, Sugano et al (1991) made a recommendation as shown in Equation 4.3.

$$depth \geq \frac{f_y d}{11\sqrt{f_c'}} \quad (4.3)$$

where

f_y = the steel yield strength

d = the column depth

f_c' = the concrete strength

4.2.3 JOINT STRENGTH

For code provision of joint strength, the diagonal strut model was widely used. The governing strength can be either a diagonal compressive strength (i.e. proportional to f_c') or a diagonal tensile strength (i.e. proportional to $\sqrt{f_c'}$). While the Standards

of Association of New Zealand (NZS 3101:1995) adopts a diagonal compressive strength approach, ACI-318-95 adopts a diagonal tensile strength approach.

$$\text{NZS 3101:1995} \quad 0.2f_c' \quad (\text{MPa}) \quad (4.4a)$$

$$\text{ACI-ASCE 352:} \quad 1.0\sqrt{f_c'} \quad (\text{MPa}) \quad (4.4b)$$

Experiments performed on specimens designed non-seismically according to BS 8110, representative of non-seismic designs in Singapore, showed that the maximum joint stress attained was $0.11f_c'$ or $0.61\sqrt{f_c'}$ (Li et al, 2002; Wu et al, 2001; Li et al, 2003).

Implications

As can be seen, direct adoption of the joint shear strength for seismic designs for structural design in non-seismic regions is not conservative owing to a lower joint strength in the latter.

4.3 FAILURE

RC members being a composite have its load resisting mechanism as an integration of the components of concrete and reinforcement. Arising from this load transfer mechanism, failure can exist on different levels being

- a. Member failure,
- b. Joint failure and
- c. Structural failure,

which shall be discussed in this section

4.3.1 MEMBER FAILURE (CHUNG ET AL, 1987)

A summary of member failure modes was made by Chung et al (1987) as listed below and illustrated Figure 4.3.

Failure Modes

1. Flexural failure from the crushing of concrete in compression following significant yielding of tensile rebar
2. Flexural failure from the fracture of the tensile reinforcement owing to large plastic deformations
3. Flexural failure from the crushing of concrete before yielding of tensile rebar
4. Flexural failure from the buckling of longitudinal rebar following the spalling of concrete cover
5. Flexure-shear failure triggered by the vertical flexural cracks which upon load reversals, cause shear failure
6. Web-shear failure from diagonal cracking
7. Web-shear failure from yielding of transverse reinforcement owing to the reduced dowel action as concrete cover spalls
8. Bond failure

For failure mode 5, the concrete shear strength contribution would outweigh the transverse reinforcement contribution owing to the non-diagonal crack angle.

Figure 4.3 - Failure modes in members (Chung et al, 1987)

4.3.2 JOINT FAILURE

Several possible modes of joint failure are as follow:

- a. failure of the diagonal strut (either in compression or tension)
- b. failure of the transverse reinforcement
- c. bond failure

While modes (a) and (b) arise from joint strength exceedance by the diagonal strut mechanism and the truss mechanism, respectively, mode (c) arises from bond strength exceedance.

For a non-seismically designed subassembly, it was observed that joint failure occurred when the drift ratio was 3% of the storey height (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003).

4.4 POST-ELASTIC BEHAVIOR

Better post-elastic modeling was the impetus to hysteretic modeling development. Hysteretic modeling as suggested by its name models the post-elastic behavior via describing the dissipation of hysteretic energy. When a member deforms, energy from the excitation is transferred to the member in the form of potential energy. At the state of maximum deformation, this potential energy is the area under the loading branch of the load-deflection curve. Figure 4.4 shows the load-deflection curve of a perfectly elastic and an inelastic single-degree of freedom (SDOF) system under ground motion.

When reaching the maximum positive deflection, the potential energy stored within the elastic SDOF system is the triangular area and that in an inelastic SDOF system is the trapezoidal area (Figure 4.4). As the SDOF system returns to its initial position, part or all of the potential energy is converted into kinetic energy, which is the area under the unloading branch of the load-deflection curve. In elastic systems, the potential energy is completely converted into kinetic energy (Figure 4.4). In the inelastic counterparts, partial conversion of the potential energy into kinetic energy is experienced, which is the triangular area in Figure 4.4. The remaining of the potential energy is dissipated by the system and known as hysteretic energy.

As damage propagates, the amount of hysteretic energy dissipated would decrease with each load cycle, where the decrease is used by hysteretic modeling to describe the post-elastic behavior.

This section shall review the various modeling techniques that have been adopted for RC members and also those proposed for joints.

Figure 4.4 - Hysteretic energy in elastic and inelastic systems

In an RC member, the hysteretic energy is influenced by several factors as has been observed from experiments (Park et al, 1972; Otani and Sozen, 1972; Kutsu and Bouwkamp, 1973; Atalay and Penzien, 1975) including:

1. Concrete strength
2. Extent of concrete confinement by transverse reinforcement
3. Amount of longitudinal reinforcement
4. Anchorage of the reinforcement
5. Amount and the spacing of the shear reinforcement
6. Load history
7. Moment-to-shear ratio or shear span

8. Magnitude of the axial force

Several hysteretic models have evolved including:

- a. Bilinear Model
- b. Trilinear Model
- c. Three-Parameter Park and Ang Model (Park and Ang, 1985; Park et al, 1985)

Variations of each model reflect the extensiveness of the model's ability to reduce the hysteretic energy during a cyclic response through mechanisms such as:

1. the change in stiffness as the member enters the post-elastic region
2. the loss of stiffness during reloading arising from cracking of concrete and/or yielding of concrete and steel (*stiffness degradation*)
3. the loss of strength in subsequent cycles when the strength is reached (*strength deterioration*)
4. the bond slip of rebars causing an abrupt change in the deformation during load reversals (also known as the *pinching effect*)

4.4.1 THREE-PARAMETER MODEL OF PARK AND ANG (1985)

In 1985, the three-parameter Park and Ang's model was proposed. Hysteretic controlling parameters were used to describe post-elastic behavior in members. These hysteretic controlling parameters are as follows:

1. Stiffness degrading parameter (α)
2. Strength deterioration parameter (β) and
3. Pinching parameter (γ)

This hysteretic model has since grown in popularity due to its versatility.

Stiffness degrading parameter

Where load reversals are expected, the member stiffness is reduced owing to material non-linearity and other factors such as cracking and yielding. A stiffness degradation parameter, α , was introduced for this effect. Mathematically, unloading

would target a point on the other side of the load-axis, which is α times of the yield strength, until it reaches the deformation-axis (Figure 4.5). While small values of α represent a severe case of stiffness degradation, large values represent nominal stiffness degradation (Table 4.2).

Strength Deterioration Parameter

Over load cycles, strength reduction would be experienced with the yielding of the concrete and/or reinforcement. To model this, a strength deterioration parameter, β , was introduced which controls any incremental hysteretic energy from the increased deformation. Mathematically, β gives the ratio of a normalized incremental damage to a normalized incremental hysteretic energy, dE , (Figure 4.5).

$$\beta = \left(\frac{d\delta_m}{\delta_u} \right) \div \left(\frac{dE}{\delta_u P_y} \right) = \frac{d\delta_m P_y}{dE} \quad (4.5)$$

where

δ_m = maximum deformation under ground motion

δ_u = ultimate deformation under monotonic load

dE = incremental hysteretic energy

P_y = yield strength

While smaller β values would allow more incremental energy and so representing less strength deterioration, larger β values represent severe strength deterioration (Table 4.2).

Pinching Parameter

Pinching is seen when a large deformations during reloading of a small load value is experienced and arises from either bond deterioration or cracks induced from previous loading cycles. This behavior is incorporated via a pinching parameter, γ . During reloading, the load path targets γP_y , until a crack closing point is reached and so reducing the hysteretic energy (Figure 4.5). While small γ values portray severe pinching effect, a value of 1.0 would represent no pinching since reloading now targets the yield strength (Table 4.2).

Chapter 4: Literature Review – Phase II of EIGM

Through the interaction of the three parameters of α , β and γ a more versatile post-elastic behavior of concrete members can be achieved.

Table 4.2 - Typical range of values for hysteretic modeling

Parameter	Meaning	Value	Effect
α	Stiffness degrading	0.1	Severe degradation
		2.0	Nominal degradation
		10.0	Negligible degradation
β	Strength deterioration	0.0	No deterioration
		0.1	Nominal deterioration
		0.4	Severe deterioration
γ	Pinching	0.1	Extreme pinching
		0.5	Nominal pinching
		1.0	No pinching

Figure 4.5 - Parameters in Park and Ang's three-parameter hysteretic model (Park and Ang, 1985)

4.4.2 JOINT POST-ELASTIC BEHAVIOR

Traditionally, the analytical models used for structures subjected to earthquakes do not include the deformation of beam-column joints. This means that the beams and columns meeting at a joint maintain a 90° angle. Researchers have recently contributed towards relaxing this rigid-joint assumption.

Ghobarah and Biddah (1999)

Here, the joint model was flexible and relaxed the rigid joint assumption. The joint model was modeled as a dimensionless joint that can undergo inelastic deformation. The joint was represented by two rotational springs that had post-elastic behavior.

Chapter 4: Literature Review – Phase II of EIGM

Chung et al (1988) integrated the joint element into an analytical software known as Seismic Analysis of Reinforced Concrete Frames (SARCF). In the comparison between the flexible joint and the rigid joint, two three-storey RC frames were subjected to a similar earthquake. While one RC frame had the flexible joints, the other had rigid joints. The earthquake used had a peak ground acceleration of 0.3 g. In the case of the RC frame with the flexible joints, the roof displacement ranged between -80 mm to 60 mm. On the other hand, the RC frame with the rigid joints experienced a roof displacement between -60 mm to 60mm.

Lowes et al (2004a, 2004b)

A flexible joint model was developed with the characteristics of stiffness degradation, strength deterioration and pinching. Unlike most dimensionless joints, this joint had a finite dimension. This joint model was recently included in the analytical software called Open System for Earthquake Engineering Simulation Version 1.6.0, OPENSEES (Mazzoni, 2004). For an interior joint, which means being connected to two beams and two columns, the joint model comprised:

- 4 external nodes that link the joint model to the nodes of beams and columns.
- One shear-panel component, eight bar-slip components and four interface-shear components.

This joint model had deformation characteristics such as shear panel action.

Correlation of the post-elastic hysteretic parameters was performed using experimental results from the two sets of experimental cyclic testing of beam-column subassemblages by Stevens et al (1991). Of the two sets of experimental results, the joint model only produced analytical response that was significantly similar to only one set of the experimental results.

Implication

It can be seen that flexible joint models allows for more accurate results. At the same time, the available joint models do produce results that may not match

experimental results. Related to the joint failure of a structure subjected to EIGM, adopting experimental results may be more appropriate. Furthermore, the joint model developed by Lowes et al (2004a, 2004b) is based on RC details for seismic-resistant design, which is not applicable in the scope of the thesis. Thus, in this thesis, a 3% inter-storey drift based on experimental observations shall be used as an indication of joint failure, which is for the non-seismically designed joints in Singapore buildings (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003).

4.5 DAMAGE ASSESSMENT

Damage assessment has been designed to provide qualitative information of structural performance under earthquakes, where some are reviewed here.

4.5.1 DAMAGE ASSESSMENT: PROCESS

Communities concerned with structural performance under earthquake have been presented damage assessment techniques for the purpose of a qualitative indication of the level of damage. Damage assessment formulation comprises a two-stage process whereby a proposed damage indexing system precedes the transformation of this index into qualitative damage levels. The second stage is performed based on statistical observations of structural damage following earthquakes. Thus, formulation of a robust damage assessment technique requires:

1. Condition of the structures concerned (i.e. design philosophy employed, as-built condition and condition at time in question)
2. Available ground motions
3. Visual inspections of the structure (i.e. following earthquake for the qualitative damage induced)
4. Result of in-situ non-destructive tests for mechanical properties of materials/members
5. Response records of structures (requiring measurements of structural response)
6. Structural analysis

Although serving as useful qualitative indications, damage assessment techniques have inherent limitations. Such limitations stem from the inability of the damage models to holistically describe the following:

- a) mechanics of materials of the structure
- b) varying intensity, frequency and duration of future earthquakes
- c) difference in geographical location between the location of the structure concerned and the location where development of the damage assessment technique was undertaken (i.e. transformation between damage index and observed damage level)
- d) damage already present in a structure subjected to previous earthquakes or for that matter other loads such wind loads and/or EIGMs

While development in advanced structural mechanics can address the limitation (a), using a larger database can address limitations (b) and (c).

4.5.2 DAMAGE ASSESSMENT: TYPES

In keeping with the development of damage assessment techniques, summaries of damage indices formulation have been made (Chung et al, 1987; Bracci et al, 1989; Williams and Sexsmith, 1995; Ghobarah et al, 1999). With more damage assessment techniques formulated, classification has also been made (Sozen, 1981). The damage index formulations have been broadly grouped as follows, depending on the factors incorporated in their formulation:

- a) strength-based damage indices
- b) response-based damage indices
  - a. maximum response
  - b. cumulative damage
  - c. maximum response and cumulative damage

The two major groups are strength-based and response-based damage indices. The response-based damage indices can be further categorized depending on their derivation which may be maximum response, cumulative damage or a combination of both.

Strength-Based Damage Index Aoyama, 1981

Following the 1968 Tokachioki earthquake, moderate damage was observed in about fifteen percent of the code-designed RC structures, with column shear failure considered as a predominant damage case. With a predicted magnitude 8 earthquake in the Tokai area within the 20th century by seismologists in Japan, effort was undertaken to propose a damage assessment technique. With the observed shear damage and failure in many RC frames, a damage index termed the basic seismic index was proposed by Aoyama (1981). A seismic index, I_s , is used as an indication of the damage, where its determination is performed by first calculating a basis seismic index, E_o . The formulation is shown in Equation 4.6.

$$E_o = \phi CF \quad (4.6a)$$

$$\phi = \frac{2(2n+1)}{3(n+i)} \quad (4.6b)$$

$$F = \sqrt{2\mu - 1} \quad (4.6c)$$

where

ϕ = a storey index

C = a strength index taken as a ratio of storey shear to total weight of building above

F = a function of the ductility index

n = number of degrees of freedom

i = storey level

μ = ductility

$$I_s = E_o GS_D T \quad (4.6d)$$

where

G = geographical index to account for local ground conditions (taken as 1)

S_D = structural design index to account for the level of symmetry in the building and the complexity of structural layout (ranges from 1.2 to 0.4)

T = time index to account for the loss of strength and age of the Building (ranges from 1.0 to 0.5)

Chapter 4: Literature Review – Phase II of EIGM

The proposed seismic index was applied to seven buildings that were exposed to the Tokachioki earthquake, where the extent of the buildings' damage levels induced were observed. Then, the basic indices were transformed to qualitative damage levels based on the damage observations, which is shown in Table 4.3.

Table 4.3 - Seismic index and damage

Seismic Index	Damage
< 0.5	Medium to Major
> 0.7	No Damage

Implications

Thus, a modified form of this strength-based damage assessment may be applicable to the damage assessment during Phase I when structures subjected to EIGM. This is due to the large induced shear forces during Phase I. However, the strength-based damage index by Aoyama (1981) does not consider the strength-enhancement due to strain rate effect. Thus, it is not directly applicable for structures subjected to EIGMs.

Response-Based Damage Index

The earliest family of damage index is ductility-based owing to the identification of the need for a structure to undergo post-elastic deformations during an earthquake. Various ways that damage can be related to post-elastic deformation include:

- a) describing the maximum response
- b) describing the cumulative damage
- c) combining both the items into a single index

Maximum Response

Damage indices that relate the structural damage to the maximum post-elastic deformation include:

- a. Ductility Ratio (Newmark and Rosenblueth, 1971)
- b. Interstorey Drift (Paulay and Priestley, 1992)

Chapter 4: Literature Review – Phase II of EIGM

- c. Flexural Damage Ratio (Lybas and Sozen, 1977; Banon et al, 1980; Roufaiel and Meyer, 1981)

DUCTILITY RATIO

As early as the 1970s, the ductility ratio was proposed by Newmark and Rosenblueth (1971), where its use as a damage index commenced. The ductility ratio, μ , is a ratio of the maximum deformation to the yield deformation, where the deformation can be in terms of displacement or curvature ductility ratio. While offering simplicity, studies have shown that it may lack consideration of any joint shear distortion arising from bond slip (Ayala and Xianguo, 1995). This implies that it may not be suitable for EIGMs, where high joint shear can be expected.

FLEXURAL DAMAGE RATIO

The flexural damage ratio discussed here is by Roufaiel and Meyer (1981). The modified flexural damage ratio (MFDR) was proposed by Roufaiel and Meyer (1981), defined as a ratio of the initial elastic stiffness to the secant stiffness at the onset of failure as shown in Equation 4.7 and illustrated in Figure 4.6. Provisions have been made for bidirectional rotation.

$$MFDR = \max[MFDR^+, MFDR^-] \quad (4.7a)$$

$$MFDR^+ = \frac{\frac{\phi_x^+}{M_x^+} - \frac{\phi_y^+}{M_y^+}}{\frac{\phi_m^+}{M_m^+} - \frac{\phi_y^+}{M_y^+}} \quad (4.7b)$$

$$MFDR^- = \frac{\frac{\phi_x^-}{M_x^-} - \frac{\phi_y^-}{M_y^-}}{\frac{\phi_m^-}{M_m^-} - \frac{\phi_y^-}{M_y^-}} \quad (4.7c)$$

where

- ϕ_y^i = yield curvature in i -direction
 ϕ_x^i = arbitrary curvature in i -direction
 ϕ_m^i = ultimate curvature in i -direction
 M_y^+ = yield moment in i -direction

Chapter 4: Literature Review – Phase II of EIGM

M_x^+ = yield moment in i -direction

M_m^+ = ultimate moment in i -direction

Following that, the formulation of a global damage parameter (GDP) was developed (Roufaiel and Meyer, 1987).

$$GDP = \frac{d_R - d_Y}{d_F - d_Y} \quad (4.8)$$

where

d_R = maximum roof displacement

d_Y = roof displacement at the yield of the first member

d_F = roof displacement at the failure of the structure

Figure 4.6 - Illustration of MDRF (Roufaiel and Meyer, 1987)

While the yield roof displacement is based on the first mode of the deformation, experimental results have suggested that an estimate of 6% of the building height can be used (Roufaiel and Meyer, 1987).

The MDRF has the added feature of describing load reversals, as compared to the ductility ratio, since it considers the possible flexural rotational direction.

Review of these damage indices shows that there is no consideration of the energy accumulation and damage accumulation. Hence, damage arising from the intensity, duration and frequency of an earthquake (or any other ground motions) may not be captured. Furthermore, they do not include the pinching effect suggesting that they may be unsuitable for cases with bond slip.

Cumulative Damage

This family of damage indices being response-based was motivated by the need to capture the cumulative damage and so can:

- a. reflect the cumulative damage
- b. describe the intensity, duration and frequency of the earthquake vital to cyclic loadings

Cyclic loading of different characteristics may induce different damage levels in various failure modes. From observations of RC structural damage following an earthquake, cyclic loading leads predominantly to failure resulting from bond slip and shear degradation rather than purely flexural failure (Paulay and Priestley, 1978). Thus, the effect of loading sequence should be captured within a damage index (Figure 4.7).

Figure 4.7 - Cyclic loads of three different load sequence

The cumulative damage indices hinges mainly on Miner's postulate (1945). Miner postulated that damage for steel is related to the accumulation of the stress levels

Chapter 4: Literature Review – Phase II of EIGM

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above the failure stress. As experimental studies showed that it may not be directly applied to RC, modifications were made leading to damage indices such as

- a. Low Cycle Fatigue (Stephens and Yao, 1987)
- b. Normalized Cumulative Rotation (Banon et al, 1980)

LOW CYCLE FATIGUE

Damage due to cyclic loading was postulated by Yao and Munse (1962), where a damage assessment formulation was proposed, grounded in cyclic axial tests was conducted on mild steel. This index seeks to relate the damage to the cumulative effect of the plastic strain at the extreme fiber.

Further modification of this was made for RC (Stephens and Yao, 1987) and as shown.

$$Damage = C \sum_1^n \left(\frac{\Delta \delta_p^+}{\Delta \delta_f} \right)^{1-br} \quad (4.9)$$

where

$\Delta \delta_p^+$ = incremental positive plastic deformation during i^{th} cycle

$\Delta \delta_f$ = maximum change in plastic deformation to cause failure

$r =$ relative plastic strain ratio (i.e. $\frac{\Delta \delta_p^+}{\Delta \delta_p^-}$)

$b =$ deformation coefficient ratio (taken as 0.77 for RC structures)

$C =$ damage acceleration coefficient (taken as 1.77)

$n =$ number of structural members

This model's limitations stem from the difficulty involved in determining $\Delta \delta_f$ and the statistical variance of the deformation coefficient ratio, b , for a composite like RC. Although able to capture the effect of loading cycle, this damage index family does not provide sufficient ductility information. This may be a shortcoming in view of the popular views on energy being the damaging potential of earthquakes, prompting the unification of maximum deformation and cumulative damage indices.

Chapter 4: Literature Review – Phase II of EIGM

Maximum Deformation and Cumulative Damage

Paradigm shift towards the energy school of thought was an impetus to capture the demand of an earthquake and so commencing research to relate the damage to demand. Such a formulation of damage index would then incorporate the cumulative damage as well as the maximum deformation.

Some of these damages indices include:

- a. Work Index
- b. Normalized Dissipated Energy
- c. Normalized Energy Index
- d. Park and Ang's Damage Index

PARK AND ANG'S DAMAGE INDEX (1985)

This damage index was proposed by Park and Ang and relates the damage to the maximum deformation and the hysteretic energy. Linear superposition was adopted in combining damage arising from the maximum deformation and the hysteretic energy. The superposition involved an empirical factor, β , which included influencing factors such as:

a. Shear span ratio,	b. Axial force.
c. Confinement effectiveness and	d. Flexural stiffness

$$D = \frac{\delta_{max}}{\delta_u} + \frac{\beta}{Q_y \delta_u} \int dE \quad (4.10a)$$

where

δ_{max} = maximum deformation experienced at time t

δ_u = ultimate deformation during monotonic loading when concrete reaches ultimate strain

Q_y = calculated yield strength

dE = dissipated energy increment

Chapter 4: Literature Review – Phase II of EIGM

$$\beta = \left(-0.447 + 0.73 \frac{l}{d} + 0.24n_0 + 0.34P_t \right) 0.7^{\rho_w} \quad (4.10b)$$

where

δ_{max} = maximum deformation experienced at time t

δ_u = ultimate deformation during monotonic loading when concrete reaches ultimate strain

Q_y = calculated yield strength

dE = dissipated energy increment

$\frac{l}{d}$ = shear span ratio

n_0 = normalized axial force

ρ_w = confinement ratio

P_t = longitudinal steel ratio

Terms included in the linear combination were based on extensive experimentation conducted in Japan (Short Column Committee, 1973-1977), where some observations were:

- a. Columns failing in shear during cyclic loadings
- b. Failure caused by bond slip occurring at increasing load cycles took place at a lower deformation than suggested by monotonic loading

The determination of the empirical constants like β was based on a regression analysis of results from earlier experimental results (Burns and Siess, 1962; Celebi and Penzien, 1973; Hirosawa et al, 1973; Kutsu and Bouwkamp, 1973; Wight and Sozen, 1973; Atalay and Penzien, 1975; Hirosawa, 1975; Newmark et al, 1975). In 1986, a transformation of the damage index to qualitative damage levels was proposed as shown in Table 4.3 (Park et al, 1985).

With the development of this group of damage indices, damage can then be better portrayed. Nevertheless, the damage only incorporates a shear span ratio to describe the effects of shear force. Thus, shear force exceedance is not explicitly described.

Chapter 4: Literature Review – Phase II of EIGM

Table 4.4 - Transformation of damage index to damage levels

Degree of Damage	Physical Appearance	Damage Index	State of Building
Collapse	Partial or total collapse of building	> 1.0	Building Loss
Severe	Extensive crushing of concrete; disclosure of buckled reinforcement	$0.4 - 1.0$	Beyond Repair
Moderate	Extensive large cracks; spalling of concrete in weaker elements	< 0.4	Repairable
Minor	Minor cracks; partial crushing of concrete in columns		
Slight	Sporadic occurrence of cracking		

4.6 IMPLICATIONS

Below are the implications from this section which are related to joint shear strength capacity, damage assessment and joint failure.

4.6.1 JOINT DIAGONAL SHEAR CAPACITY

Whether it is column strength or joint strength, strength adequacies are required in order to resist EIGMs, especially in view of the greater shear forces developed. For the diagonal shear strength capacity, Priestley's predictive strength equation shall be used in the analytical models to follow (Priestley et al, 1994).

4.6.2 DAMAGE ASSESSMENT

From the damage assessment review, while the strength based-damage indices can show strength exceedance, the response-based damage indices can show damage accumulation. For EIGMs, Phase I response may display shear exceedance which may not be captured correctly using a response-based damage index alone. While using a strength-based damage index may display the possible failure due to high

shear in Phase I, the damage accumulated during Phase II response may not be properly reflected. Thus, damage assessment of EIGM effects may require a combination of the strength-based and the response-based damage indices.

4.6.3 JOINT FAILURE

Furthermore, joints being a critical region would also require attention during damage assessment as it can contribute significantly to overall structural damage assessment for non-seismically designed structures. Here, a 3% inter-storey drift shall be used as an indication as joint failure, based on the experimental observations for non-seismically designed joints in Singapore (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003).

5 TIMOSHENKO BEAM VS. EULER-BERNOULLI BEAM

This section deals with the numerical investigation of:

- a) Determining the response of columns induced by a triangular EIGM
- b) The effect of boundary conditions on the response.
- c) The effect of using different beam theories on the response determined

To achieve this, the following was considered:

- Two types of boundary conditions were used for the columns to study the effect of boundary conditions on the response of columns subjected to EIGMs. While one column has a fixed-fixed boundary condition, the other has a fixed-free boundary condition.
- For the study of the effect of beam theories on the response of columns subjected to EIGMs, the beam theories used are the Timoshenko beam theory and the Euler-Bernoulli beam theory. Finite element modeling of the two beam theories is applied. Both the finite element modeling of the Euler-Bernoulli beam theory (EBT-FEM) and the finite element modeling of the Timoshenko beam theory (TBT-FEM) are applied for column of different boundary conditions.

5.1 ANALYTICAL MODEL AND EIGM USED

Figure 5.1 shows the fixed-fixed column and the fixed-free column for the comparison of the two beam theories. For the case of the fixed-free column, a mass of 5177 kg is added at the top of the column. This added mass represents the total mass of the structure above. Figure 5.3 shows the acceleration time history of the single triangular EIGM load that is used. The triangular EIGM load is applied in the direction parallel to the depth of the column, which is 300 mm. This triangular EIGM has the following characteristics:

- Peak ground acceleration (PGA) of 80 g
- Duration, $t_d = 2.5$ ms
- Peak ground velocity or peak particle velocity (PPV) of 1 m/s
- Principal frequency of 200 Hz

Chapter 5: Timoshenko Beam vs. Euler-Bernoulli Beam

For the triangular load (Figure 5.1a), the principal frequency is given in Equation 5.1a and the relationship between PGA, PPV and duration is given in Equation 5.1b.

$$\text{Principal frequency} = \frac{1}{t_d} \quad (5.1a)$$

$$\text{PPV} = 0.5 \times \text{PGA} \times t_d \quad (5.1b)$$

where

$$t_d = \text{duration of the EIGM}$$

The response analysis that is performed is an elastic analysis that uses the direct integration method. The acceleration in the direct integration method is considered to be constant between each time step. The time steps used were at equal intervals of 2.5E-4 ms. The RC section is treated as a composite section, where the effective moment of inertia of the composite section of steel reinforcements and concrete is adopted for the response analysis.

Figure 5.1 - EIGM used and models of column used

5.2 FIXED-FIXED COLUMN

In the case of the fixed-fixed column, several types of finite element models were used, which are as follows:

- A TBT-FEM model of the fixed-fixed column using 19 elements (TBT-19)
- An EBT-FEM model of the fixed-fixed column using 19 elements (EBT-19)
- An EBT-FEM model of the fixed-fixed column using 12 elements (EBT-12)
- An EBT-FEM model of the fixed-fixed column using 4 elements (EBT-4)

The objective is to investigate the following:

- The difference in response results between using the EBT and TBT
- The effect of increasing the number of elements for EBT-FEM

5.2.1 MODE SHAPES

Some mode shapes for the TBT-19 model of the fixed-fixed column is shown in Figure 5.2. The mode shapes are plotted as normalized height y/h against the orthonormalized displacements. As can be seen, the local modes vibrations have a high modal frequency. Some of the modal frequencies for the different models of the fixed-fixed column are summarized in Table 5.1. Some observations are as follows:

- For different beam theories with the same number of elements (i.e. EBT-19 and TBT-19), TBT-FEM produces a lower modal frequency than those by EBT-FEM. This *softer* nature determined by TBT-FEM agrees with the results by other researchers (Antes et al, 2004).
- For the same number of elements used, TBT-19 has more modes than the EBT-19 due to the rotatory inertia.
- For the EBT models, decreasing the number of elements used leads to decreasing modal frequencies. This can be seen by comparing the modal frequencies of the EBT-19, EBT-12 and EBT-4 models.

Figure 5.2 - Selected mode shapes of TBT-19 for fixed-fixed column

Table 5.1 - Modal frequencies of fixed-fixed column

	Mode 1	Mode 2	Mode 3	Mode 4	Mode n^*
TBT-19	89 Hz	244 Hz	479 Hz	793 Hz	1.7 MHz
EBT-19	89 Hz	245 Hz	481 Hz	796 Hz	0.2 MHz
EBT-12	89 Hz	244 Hz	479 Hz	790 Hz	3.9 kHz
EBT-4	88 Hz	234 Hz	-	-	386 Hz

* Note: n denotes the highest mode

5.2.2 TBT-19 vs. EBT-19

This section compares the response results for two beam theories with the same number of elements used.

Relative Displacement and Rotation

The time histories for relative displacement and rotation for EBT-19 and TBT-19 are shown in Figure 5.3. The time histories are plotted against the normalized time t/t_d . Table 5.2 shows a summary of the maximum absolute values of the response at certain points along the column length and the time at which they are attained.

From Figure 5.3, the following observations can be made:

- For both models, the relative displacement is the highest at mid height of the column ($y/h = 0.5$).
- Both models produced relative displacement time histories that are similar
- For both models, the rotation at $y/h = 0.25$, the relative displacement time histories as determined were similar.
- However, at the mid-height ($y/h = 0.5$), the maximum absolute rotation is higher in the case of EBT-19 than in TBT-19. This can be accounted for by the presence of rotatory inertia in TBT-19 as compared to EBT-19. Further investigation would reveal if this impacts on the induced shear forces.

Chapter 5: Timoshenko Beam vs. Euler-Bernoulli Beam

Figure 5.3 - Displacement and rotation time histories for EBT-19 and TBT-19 of fixed-fixed column

From Table 5.2, the following observations can be made:

- Along the entire column length, the peak relative displacements as determined using both the EBT-19 and TBT-19 model occurs at the end of the loading duration ($t/t_d = 1.0$).
- Along the entire column length, the relative displacements as determined by the TBT-19 model are slightly higher than that by the EBT-19 model.

Table 5.2 - Summary of peak relative displacements and time of occurrence for EBT-19 and TBT-19 of fixed-fixed column

y/h	TBT-19	EBT-19
0.08	0.1142 mm ($t/t_d = 1.0$)	0.1130 mm ($t/t_d = 1.0$)
0.25	0.7691 mm ($t/t_d = 1.0$)	0.7643 mm ($t/t_d = 1.0$)
0.50	1.483 mm ($t/t_d = 1.0$)	1.482 mm ($t/t_d = 1.0$)

Induced Shear Forces and Bending Moments

Figure 5.4 shows the time histories for the induced shear forces and induced bending moment for both models. For both the EBT-19 and TBT-19 models, the distributions of induced shear forces along the column length at the time when the induced shear forces at the base are maximums, are also shown (Figure 5.4). For both models, the maximum absolute shear forces along certain points of the column and the time at which these maxima occur are summarized (Table 5.3).

From Figure 5.4, the following observations can be made:

- For both models, the induced shear force is larger at the base of the column ($y = 0.0h$) than at the top of the column ($y = 1.0h$). The induced shear forces determined by both models are similar. At the base, the induced shear force reaches its peak value before the end of the loading duration ($t/t_d < 1$). This is due to the effect of large inertia forces during the loading duration ($t/t_d < 1$).

Chapter 5: Timoshenko Beam vs. Euler-Bernoulli Beam

- For both models, the induced bending moment at the base of the column is similar to that at the top. Both models produce similar bending moments. At the base, the induced bending moment reaches its peak value at the end of the loading duration ($t/t_d = 1$).
- At the time when the induced shear force is the largest at the base, both models produce similar distribution of shear forces along the column height (Figures 5.4c and 5.4d).

Table 5.3 -Summary of shear forces and bending moment for TBT-19 and EBT-19 models of fixed-fixed column

y/h	Shear Forces (kN)		Bending Moment (kNm)	
	TBT-19	EBT-19	TBT-19	EBT-19
0.00	95.6 ($t/t_d = 0.63$)	95.1 ($t/t_d = 0.61$)	42.0 ($t/t_d = 1.0$)	41.5 ($t/t_d = 1.0$)
0.50	0.6 ($t/t_d = 0.90$)	0.3 ($t/t_d = 0.98$)	28.2 ($t/t_d = 1.0$)	28.1 ($t/t_d = 1.0$)
1.00	83.8 ($t/t_d = 0.65$)	83.4 ($t/t_d = 0.64$)	41.7 ($t/t_d = 1.0$)	41.9 ($t/t_d = 1.0$)

Chapter 5: Timoshenko Beam vs. Euler-Bernoulli Beam

Figure 5.4 - Induced shear forces and induced bending moments for EBT-19 and TBT-19 of fixed-fixed column

Closer inspection of Table 5.3 leads to the following observations:

- The induced shear forces along the height of the column are slightly larger as determined by the TBT-19 model than the EBT-19 model. Along corresponding points for both models, the induced shear forces reach their maximum absolute values in the TBT-19 model slightly later than the EBT-19 model.
- The induced bending moments at the top of the column ($y = 1.0h$) is slightly smaller as determined by the TBT-19 model than the EBT-19 model. Along corresponding points along the height of the column, the induced bending moments reach their maximum absolute values at the end of the loading duration ($t/t_d = 1$) for both models.
- Nevertheless, the induced shear forces and induced bending moments as determined by both models are practically similar.
- The above three observations can be explained as follows. For an element of EBT, it has been said that the wave propagates at infinite speeds as compared to finite speeds for the case of TBT. However, by discretizing a Euler-Bernoulli continuum into finite elements, the results are similar to the discretized form of a Timoshenko continuum. During the finite element modeling of the fixed-fixed column, the mass along the column was distributed at the nodes between the elements. As such, these masses act as filters leading to the blocking of higher frequency waves from propagating further along the height of the column. This agrees with the observations by Doyle (1997) and Lu et al (2001). To rephrase, it is true that waves can propagate at infinite speeds along one element of EBT. However, when the element is further discretized, a mass is present between *two* EBT elements. This mass acts as a filter to wave propagation across this mass. If we recall the d'Alembert force equilibrium, such an explanation is more comprehensible. At a node, the force equilibrium needs to be satisfied. As such, the presence of the mass at a node and the inertia forces that it would have ensured that the induced shear forces at the ends of two Euler-Bernoulli elements that meet at that particular joint are not the same. Thus, the EBT-19 produced results which are similar to the TBT-19.

Chapter 5: Timoshenko Beam vs. Euler-Bernoulli Beam

- If that is the case, why are the shear forces at the supports reaching their maximum absolute values earlier in the case of the EBT-19 model as compared to the TBT-19 model? This can be explained by the way in which the waves propagate along height of the column. Discretizing the column as finite elements of EBT and distributing the mass along the column height has introduced finite wave propagation speeds. However, due to the nature of the beam theory that is used for the elements of the EBT-19 model, the finite wave propagation speed as introduced will still be slightly larger than the wave propagation speed for the elements in the TBT-19 model. Thus, the waves propagate along the height of the column at higher speeds in the case of EBT-19 model than the TBT-19 model.
- Although the shear forces at the support reach their maximum absolute values at an earlier time for the EBT-19 model as compared to the TBT-19 model, the magnitude as determined by the TBT-19 model is slightly larger than that determined by the EBT-19 model. Resulting from a larger wave propagating speed in the EBT-19 model, the column starts to deform at an earlier time for the EBT-19 model as compared to the TBT-19 model.
- The induced shear force is largest at the base of the column for both models. While differences in the induced shear force at the mid-height of the column ($y/h = 0.50$) exists, its magnitude is very small compared to the induced shear force at the base of the column ($y/h = 0.00$).

Discussion

From the investigation here, the following is a summary of the observations and implications:

- For both models, the induced shear force and bending moment at the base of the column are the largest along the height of the column. At the base, the shear force reaches its peak value before the end of the loading duration ($t/t_d < 1$) while the bending moment reaches its peak value at time $t/t_d = 1$.
- Discretized model of EBT seems to introduce finite wave propagation speeds which are otherwise infinite for EBT continuums. However, the

effect of the number of elements used in discretizing an EBT continuum of the finite wave propagation speed should be studied.

- For the same number of elements, the EBT-FEM model produced results similar to the TBT-FEM model. However, the effect of the number of elements used for EBT-FEM models on the accuracy of the results should be studied.

5.2.3 EFFECT OF NUMBER OF ELEMENTS USED IN EBT-FEM

This section investigates the effect of the number of elements used in EBT-FEM models on the accuracy of the results. As it has been established that the shear force is the largest at the base, greater attention is paid to this. For the fixed-fixed column, EBT-FEM models with fewer numbers of elements are used for this study, which are:

- An EBT-FEM model of the fixed-fixed column using 12 elements (EBT-12) and
- An EBT-FEM model of the fixed-fixed column using 4 elements (EBT-4).

Table 5.4 compares the results of the shear forces and bending moments induced at the supports as determined by the EBT-12 model, the EBT-4 model and the earlier EBT-19 model. The observations are made as follows:

- By decreasing the number of elements from EBT-19 to EBT-12 and EBT-4, both shear force and bending moment at the base increase.
- The rate of increase is higher for shear force than bending moment.

Table 5.4 - Shear forces and bending moments for EBT models of fixed-fixed column

Shear force (kN)			
y/h	EBT-19	EBT-12	EBT-4
0.00	95.1 ($t/t_d = 0.61$)	97.2 ($t/t_d = 0.65$)	176.2 ($t/t_d = 1.00$)
1.00	83.4 ($t/t_d = 0.65$)	83.6 ($t/t_d = 0.65$)	83.7 ($t/t_d = 1.0$)
Bending Moment (kNm)			
y/h	EBT-19	EBT-12	EBT-4
0.00	41.5 ($t/t_d = 1.0$)	41.9 ($t/t_d = 1.0$)	72.8 ($t/t_d = 1.0$)
1.00	41.5 ($t/t_d = 1.0$)	41.9 ($t/t_d = 1.0$)	52.0 ($t/t_d = 1.0$)

The explanation for an increased shear force and bending moment at the base of the column when the number of elements used in EBT-FEM is decreased can be found in the displacements determined by each model. Figure 5.5 shows the displacement profile at the time when the induced shear force at the base reaches its maximum absolute value. For comparison, the displacement profile for the time when the induced shear force at the base reaches its maximum absolute value is also presented for the TBT-19 model. The following observations can be made:

- For all the models, the largest displacement occurs at the mid-height ($y/h = 0.50$). For the EBT-FEM models, as the number of elements used increases, the displacement at the mid-height reduces. When the number of elements used is decreased, the number of nodes along the height of the column is also decreased. With fewer nodes along the column, there will be fewer lumped masses along the column. As a result of the reduced number of lumped masses, the extent of the blocking of high frequency waves will also decrease. With a reduced extent of the blocking of high frequency waves by the lumped mass, the wave propagation speeds will increase. In short, for

EBT-FEM, when the number of elements used is decreased, the wave propagation speeds along the column increases, which will result in the column undergoing a larger displacement.

- As a result of increased displacements with decreasing number of elements used, the induced shear forces and bending moments will increase.

Figure 5.5 - Displacement profiles at maximum shear for fixed-fixed column

5.2.4 DISCUSSION

From the study of the fixed-fixed column subjected to a triangular EIGM load, the following has been observed:

- For both of the finite element modeling of Euler-Bernoulli beam theory and Timoshenko beam theory, the induced shear force is largest at the base of a fixed-fixed column. Along the height of the column, the displacement is the largest at the mid-height ($y = 0.5h$). This is attributed to the local mode contribution which was observed earlier (Lu et al, 2001; Pan et al, 2002; Dhakal and Pan, 2003).
- At the base, the shear force reaches its peak value before the end of the loading duration ($t/t_d < 1$) while the bending moment reaches its peak value at the end of the loading duration ($t/t_d = 1$)
- For the same number of elements used, the rotation at the mid-height of the column ($y = 0.5h$) is larger when determined by EBT-FEM as compared to TBT-FEM. This is due to the rotatory inertia that is present in TBT-FEM

which is otherwise absent in EBT-FEM. However, the displacement along the column as determined by both EBT-FEM and TBT-FEM is similar when the same number of elements is used.

- When the same numbers of elements are used for both EBT-FEM and TBT-FEM, the induced shear forces and induced bending moments determined are similar. This implies that finite element modeling of EBT can produce similar shear forces and bending moment as finite element modeling of TBT.
- While the rotation at the mid-height of the column ($y = 0.5h$) is larger in EBT-FEM than TBT-FEM, the difference in the rotation at the mid-height does not significantly affect the shear forces at the base of a fixed-fixed column subjected to EIGMs.

5.3 FIXED-FREE COLUMN

In this fixed-free column example, two types of finite element models were used. They are as follows:

- A TBT-FEM model of the fixed-free column using 19 elements (TBT-19)
- An EBT-FEM model of the fixed-free column using 19 elements (EBT-19)

The fixed-free column and the triangular EIGM load used are as shown in Figure 5.1.

The effect of decreasing the number of elements on response has been discussed in section 5.2, this will not be repeated in this section. The difference in response results between using the EBT and TBT shall be studied.

5.3.1 MODE SHAPES

Some mode shapes for the TBT-19 model of the fixed-free column are shown in Figure 5.6. The higher frequency nature of the local modes can be seen from the mode shapes. For the fixed-free column, the modal frequencies for the EBT-19 model and the TBT-19 model are shown in Table 5.5.

Chapter 5: Timoshenko Beam vs. Euler-Bernoulli Beam

Table 5.5- Modal frequencies of fixed-free column

	Mode 1	Mode 2	Mode 3	Mode 4	Mode n^*
TBT-19	2 Hz	60 Hz	187 Hz	372 Hz	0.47 MHz
EBT-19	2 Hz	62 Hz	199 Hz	416 Hz	0.26 MHz

Note: n denotes the highest mode

Figure 5.6 - Selected mode shapes for TBT-19 model of fixed-free column

5.3.2 TBT-19 vs. EBT-19

This section compares the response results for two beam theories with the same number of elements used.

Displacement and Rotation

For the fixed-free column, the displacement and rotation time histories are shown in Figure 5.7. The following observations are made:

- The displacement time histories as determined by both models are similar. Along the column, the maximum absolute displacement is the largest at a height $y = 0.67 h$, and the displacement at this position is larger in magnitude than that at the top of the column. The displacement being the largest in magnitude at $y = 0.67 h$ can clearly be seen in Figures 5.7c and 5.7d. This can be attributed to local mode contribution. In both the models, the maximum absolute displacements at heights $y = 0.67 h$ and $y = 1.00 h$ are 1.38 mm and 1.25 mm, respectively. For both models, the maximum absolute displacements occurs at time $t/t_d = 1$.
- The rotation at the height $y = 0.67 h$ as determined by the EBT-19 model is larger than that determined by the TBT-19 model. As was explained in the case of the fixed-fixed column, this is due to the presence of rotatory inertia in the TBT-19 model. The effect of this larger rotation in the EBT-19 model on the induced forces will be discussed.

Induced Shear Forces and Bending Moments

Figure 5.8 shows the time histories for induced shear force and induced bending moment as determined by both the EBT-19 model and the TBT-19 model. Some of the observations that can be made are as follows:

- It can be seen that both models produced similar time histories for the induced shear forces and bending moments. Similar to the fixed-fixed column, by using a number of elements to model the EBT and distributing the mass at intermediate points along the column, finite wave propagation

Chapter 5: Timoshenko Beam vs. Euler-Bernoulli Beam

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speeds are introduced. Thus, the EBT-19 model produces results similar to the TBT-19 model.

- For both models, the induced shear forces and bending moments are the largest at the base of the column. It can be seen that the larger rotation as determined by the EBT-19 model does not significantly affect the induced forces as determined. At the base, the shear force reaches its peak value during the loading duration ($t/t_d < 1$) while the bending moment reaches its peak value at the end of the loading duration ($t/t_d = 1$). The reason for the shear force reaching its peak earlier than bending moment is attributed to the large inertia forces during the loading duration ($t/t_d < 1$).

Discussion

Following the response analysis using the fixed-free column, some of the observations which are made are summarized as follows:

- For the same number of elements, the use of EBT-FEM produces similar results for induced shear forces and bending moments as compared to the TBT-FEM.
- Similar to the fixed-fixed column, the induced shear forces and induced bending moments are the largest at the base of the column.

With this, the effect of boundary conditions on a column subjected to EIGM loading will be discussed next.

Figure 5.7 - Displacement and rotation time histories for EBT-19 and TBT-19 for fixed-free column

Chapter 5: Timoshenko Beam vs. Euler-Bernoulli Beam

Figure 5.8 - Induced shear forces and bending moment for EBT-19 and TBT-19 of fixed-free column

5.4 EFFECT OF BOUNDARY CONDITIONS

From the response analysis of two RC columns of different boundary conditions at the top of the column, similar observations are made in terms of the induced shear forces and bending moments. It is observed that the induced shear forces and bending moment are the largest at the base of the column when subjected to the same EIGM load. In terms of the design of RC sections, the forces are key design parameters. As such, the effect of the boundary conditions on the induced shear forces and bending moments are compared between the fixed-fixed column and the fixed-free column. For this comparison, the EIGM load and the columns are the same as before (Figure 5.1). Here, the comparison is made between the TBT-FEM models of the two columns. The columns are modeled using 19 elements each. In short, the results of the analysis of the TBT-19 models for the fixed-fixed column are compared to those of the fixed-free column. Figure 5.9 shows the induced shear and bending moment results from both the fixed-fixed and fixed-free columns. The observations are made as follows:

- Both fixed-fixed and fixed-free columns show similar shear and bending moment time histories at the base of the column.
- At the base, the induced shear force reaches its peak value before the bending moment reaches its.
- Along the column, the shear force distribution is larger at the top of the column ($y = 1.0 h$) for the fixed-fixed column as compared with the fixed-free column (Figure 5.9c), which is zero. However, near to the column base ($0.0 h < y \leq 0.4 h$), the induced shear is larger for the fixed-free column than the fixed-fixed column.
- Along the column, the induced bending moments along the heights of both columns are different, except at the base of the column where the values are similar. The induced bending moment is the largest at the base of the column for both columns.

Chapter 5: Timoshenko Beam vs. Euler-Bernoulli Beam

Figure 5.9 - Comparing induced shear and bending moment for fixed-fixed and fixed-free column

5.5 DISCUSSION

From the numerical investigation performed in this section, summary of the observations is as follows:

- When a column is subjected to an EIGM, the displacements are the largest at the local modes. In the case of the fixed-fixed column, it was at the mid-height of the column. In the case of the fixed-free column, the local mode response is more significant than the global mode response. Thus, the displacement is larger around the mid-height of the column than the top of the column, which is free end.
- Although the displacement is larger around the mid-height of the column, the induced shear force and induced bending moment are the largest at the base of the column. Thus, should a column fail due to strength exceedance, it would be at the base of the column where the induced shear force and bending moment are the largest. This implies that a *toughness test* during Phase I should be conducted by inspecting the induced shear force and bending moment at the column base.
- At the base, the induced shear force reaches its peak during the loading duration ($t/t_d < 1$) while the bending moment reaches its peak at the end of the loading duration ($t/t_d = 1$). This is attributed to the large inertia forces during the loading duration ($t/t_d < 1$) that lead to large shear forces developed during the loading duration.
- Research has shown that in an Euler-Bernoulli continuum, the wave propagation speeds are infinite. On the other hand, waves propagate at finite speeds in a Timoshenko beam continuum. However, the study conducted here has shown that using a sufficient number of Euler-Bernoulli beams and distributing the mass of the column at intermediate nodes introduces a finite wave speed. As such, the response results as determined by using a sufficient number of elements for Euler-Bernoulli beam can produce results similar to those of Timoshenko beam.
- At the base of the column, the shear force and bending moment induced on a fixed-fixed column is similar to that on a fixed-free column, when subjected to the same EIGM. The induced shear force and bending moment are the

Chapter 5: Timoshenko Beam vs. Euler-Bernoulli Beam

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largest at the column base where the toughness test during Phase I should be conducted. Since similar shear force and bending moment are determined by a fixed-fixed column and a fixed-free column, a fixed-fixed column may be used to produce the approximate shear force and bending moment at the column base for the toughness test related to Phase I.

6 PROPOSED METHODS AND TECHNIQUES – PHASE I OF EIGM

This section describes the proposed development surrounding the response analysis during Phase I of a structure subjected to EIGM as follows:

- a) Proposed approximate method for response analysis,
- b) Proposed damage assessment for Phase I

6.1 PROPOSED APPROXIMATE METHOD FOR RESPONSE ANALYSIS

When frames are subjected to EIGMS, the local mode contribution is significant during Phase I. These local modes are associated with the degrees of freedom at intermediate points along the distributed mass of the member. On the other hand, this local mode contribution is less significant during Phase II. For Phase II, the response is primarily due to the contribution of global modes, which are associated with floors as main mass points. Thus, the response analysis during Phase I requires the inclusion of local modes by introducing more degrees of freedom within a structural element. This leads to a greater computational effort. Furthermore, if the response analysis for Phase I and Phase II are not separated, the high number of degrees of freedom introduced for the response analysis during Phase I would stay within the model used in Phase II. While the local mode contribution is insignificant for Phase II response, including the local mode contribution within the response analysis for Phase II leads to a large computational effort that does not justify the accuracy in the response analysis. Thus, ways of reducing the computational effort for the response analysis of a frame structure subjected to EIGMs would be to:

- Separate the response analyses for Phase I and Phase II
- Develop an approximate method for the response analysis during Phase I

This section proposes the use of impulse to approximate the response during Phase I. Further, it is shown that the approximate response at the end of Phase I can be used to calculate the subsequent response during Phase II. Thus, the response

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

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analysis for Phase II is performed separately from Phase I. This would save the computational effort for the response analysis of both Phase I and Phase II. This is the advantage of the proposed approximate method.

However, the proposed approximate method should have the following characteristics:

- It is shown in Chapter 5 and observed in previous research that structures exhibit significant local mode response when subjected to EIGMs (Lu et al, 2001; Pan et al, 2002; Dhakal and Pan, 2003). Thus, the proposed approximate method should be applied to include the local mode response. In order to do so, the approximate method should be able to be used for the response analysis of a system with multiple degrees of freedom.
- Furthermore, the study in Chapter 5 shows that the induced shear force is the largest at the base of the column when subjected to an EIGM. It also shows that this shear force reaches its peak during the loading duration ($t/t_d < 1$). Thus, in order to capture the maximum shear force, the proposed method should be able to produce a time history for the response of a structure subjected to an EIGM.

The proposed approximate method for a multiple degree of freedom system (MDOF system) is developed by first developing it for a single degree of freedom system (SDOF system). The structural damping is neglected due to its insignificance in short-duration impulsive loads as shown later. Correction factors are developed for undamped structures under triangular loads. Comparison between the proposed method of approximate response analysis and the closed-form response analysis is made.

6.1.1 PROPOSED APPROXIMATE METHOD FOR A SDOF SYSTEM

The impulse transfer method was outlined by Clough and Penzien (1993). For an SDOF system displaced under a given ground motion, a force equilibrium can be

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

expressed using d'Alembert's Principle (Figure 6.1). The force equilibrium equation is shown in Equation 6.1a.

$$m\ddot{v}_t(t) + c\dot{v}(t) + kv(t) = 0 \quad (6.1a)$$

$$\ddot{v}_t(t) = \ddot{v}_g(t) + \ddot{v}(t) \quad (6.1b)$$

where

m = the mass of the SDOF system

k = the lateral stiffness of the SDOF system

c = the damping of the SDOF system

$v(t)$ = the displacement of the SDOF system relative to the ground

$\dot{v}(t)$ = the velocity of the SDOF system relative to the ground

$\ddot{v}(t)$ = the acceleration of the SDOF system relative to the ground

$v_t(t)$ = the absolute displacement of the SDOF system

$\ddot{v}_t(t)$ = the absolute acceleration of the SDOF system

With this, arranging the terms in Equation 6.1a and integrating with respect to time during Phase I would lead to Equation 6.2, as follows:

$$m \int_0^t \ddot{v}(t) dt + c \int_0^t \dot{v}(t) dt + k \int_0^t v(t) dt = -m \int_0^t \ddot{v}_g(t) dt \quad \text{when } 0 < t \leq t_d \quad (6.2)$$

where

t = time from the start of the loading

t_d = time at the end of the Phase I

Figure 6.1 - SDOF system

Effect of Damping

It has been shown that for structures subjected to impulsive loads, the effect of damping is negligible (Karim and Salim, 1998). Before the derivation of the proposed approximate method, some preliminary numerical analyses are conducted. An SDOF system is subjected to an impulsive load, and the response is analyzed using the closed-form solutions. The impulsive load and the structure are shown in Figure 6.2. For this numerical study, a critical damping ratio of 5 % was used. The fundamental period for the structure is 1.0 s. The EIGM is a half-sine ground shock with the following characteristics: (a) peak ground acceleration of 6 g, (b) occurring over 31 ms and (c) principal frequency of about 16 Hz.

The structural response is determined and also the related terms in Equation 6.2. In Figure 6.2, the time histories are shown with the response plotted against the normalized time t/t_d . The normalized time is the ratio of the time at an instant t to the duration of the EIGM t_d . These are the observations:

- Figure 6.2c shows that that the displacement during Phase I ($t/t_d \leq 1$) is negligible as compared to Phase II ($t/t_d > 1$).
- The input energy that is absorbed by damping $c \int_0^t \dot{v}(t) dt$ is also insignificant during Phase I (Figure 6.2d).

The proposed approximate method, which is based on impulse-momentum transfer, is described as follows.

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

Figure 6.2 - SDOF system and EIGM for study of structural damping

Impulse-momentum transfer

Notably, when the ground motion occurs over a short duration, the structure experiences small deformation (Dhakal and Pan, 2003). Clough and Penzien (1993) stated that the maximum displacement is dependent on the magnitude of the

impulse applied, $-m \int_0^{t_d} \ddot{v}_g(t) dt$. When the displacement is small and the effect of

damping is negligible, an approximation can be made for the acceleration of the SDOF system relative to the ground (Equation 6.3a). By integrating the Equation 6.3a with respect to time, an approximate change in the velocity relative to ground, can be deduced (Equation 6.3b). By further integrating Equation 6.3b, an approximate change for the displacement relative to the ground, can be deduced (Equation 6.3c).

$$\ddot{v}_e(t) \approx -\ddot{v}_g(t) \quad (6.3a)$$

$$\dot{v}_e(t) - \dot{v}(0) \approx -\int_0^t \ddot{v}_g(t) dt \quad (6.3b)$$

$$v_e(t) - v(0) \approx -\int_0^t [\dot{v}_e(t) - \dot{v}(0)] dt \quad (6.3c)$$

where

$\ddot{v}_e(t)$ = approximate acceleration of the SDOF system relative to the ground

$\dot{v}_e(t)$ = approximate velocity of the SDOF system relative to the ground

$v_e(t)$ = approximate displacement of the SDOF system relative to the ground

This means that the approximate acceleration of the SDOF system relative to the ground is equal in magnitude but opposite to the direction of the ground acceleration. Also, the approximate velocity of the SDOF system relative to the ground is equal in magnitude but opposite to the direction of the ground velocity. The same can be said about the approximate displacement of the SDOF system relative to the ground.

When the initial displacement and velocity of the SDOF system is zero, the approximate displacement and velocity at time $t = t_d$ can be expressed as follows:

$$\dot{v}_e(t_d) \approx -\int_0^{t_d} \ddot{v}_g(t) dt \quad \text{when } \dot{v}_e(0) = 0 \quad (6.4a)$$

$$v_e(t_d) = -\int_0^{t_d} \left(\int_0^t \ddot{v}_g dt \right) dt \quad \text{when } v_e(0) = 0 \quad (6.4b)$$

However, approximating the velocity in this manner produces an overestimate. This is because two terms in Equation 6.2 were neglected, which are $c \int_0^t \dot{v}(t) dt$ and

$k \int_0^t v(t) dt$. To produce a more accurate approximate, correction factors shall be

developed. In order to do so, a closed-form solution needs to be developed. The EIGM load considered here is a triangular EIGM load (Figure 6.3).

Closed-Form Solution for SDOF Subjected to Triangular EIGM

In this section, the closed-form solution of an SDOF system subjected to a triangular EIMG load will first be derived. Later, this solution will be compared with the proposed approximate method. Correction factors will then be proposed to improve the approximate solutions.

For the *general triangular EIGM* shown in Figure 6.3, the exact displacement and velocity response time histories for an undamped case during Phase I (i.e. $t \leq t_d$) is presented in terms of closed-form solutions. The exact displacement and velocity response are derived via the convolution procedure as shown in Figure 6.4. The convolution procedure is described as follows:

- a) The first step describes the response of an SDOF system for the time $t \leq t_d$, when subject to the left triangular load of the EIGM for the duration $0 < t \leq \mu t_d$. The response from this is termed as $R_I(t)$, which consists of the forced vibration response for the duration $t \leq \mu t_d$ and the free vibration response for the duration $t > \mu t_d$.

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

- b) The second step describes the response of the SDOF system for time $t > \mu t_d$, when subject to the right triangular load of the EIGM for the duration $\mu t_d < t \leq t_d$. The response from this is termed as $R_2(t)$, which considers the structure as being initially at rest. $R_2(t)$ is the the response of the same SDOF undergoing forced vibration when subjected to the right triangular load of the EIGM, for the duration $\mu t_d < t \leq t_d$.
- c) Both $R_1(t)$ and $R_2(t)$ are combined for the duration $0 \leq t \leq \mu t_d$. The combined response describes the exact response of the SDOF system for time $0 < t \leq t_d$, when subjected to a general triangular EIGM load.

For an undamped SDOF with given initial displacement and initial velocity, its free vibration response have been determined (Clough and Penzien, 1993). This is given in Equation 6.5.

$$v(t - \tau) = v(\tau) \cos \omega t + \frac{\dot{v}(t - \tau)}{\omega} \sin \omega t \quad (6.5a)$$

$$\dot{v}(t - \tau) = -v(\tau) \omega \sin \omega t + \dot{v}(t - \tau) \cos \omega t \quad (6.5a)$$

where

t = time from the onset of forced vibration

τ = time at which the forced vibration ends

$v(t)$ = velocity of SDOF system at time t

$\dot{v}(t)$ = displacement of SDOF system relative to the ground at time t

ω = angular frequency of the SDOF system

Having shown that contribution from damping is negligible when a structure is subjected to impulsive loads, Equation 6.1 is rewritten as follows:

Undamped equation of motion:

$$m\ddot{v}(t) + kv(t) = -m\ddot{v}_g(t) \quad (6.6)$$

Then, the triangular EIGM load is written as shown below.

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

EIGM load:

$$\ddot{v}_g(t) = A_g \frac{t}{\mu t_d} \quad (0 \leq t \leq \mu t_d) \quad (6.7a)$$

$$\ddot{v}_g(t) = \frac{A_g}{\alpha} \left(1 - \frac{t}{t_d} \right) \quad (\mu t_d < t \leq t_d) \quad (6.7b)$$

A_g = the peak ground acceleration

μ = the ratio of time where the apex of triangle is to t_d

α = $1 - \mu$

The EIGM load is substituted into Equation 6.6 and the second-order, non-homogenous differential equation is solved following the convolution procedures outlined in Figure 6.4.

Velocity:

$$\dot{v}(t) = \frac{A_g}{\omega^2 t_d} \left[\frac{\cos \omega t}{\mu} - \frac{1}{\mu} \right] \quad (0 \leq t \leq \mu t_d) \quad (6.8a)$$

$$\dot{v}(t) = \frac{A_g}{\omega^2 t_d} \left[\frac{\mu + \alpha \cos \omega t - \cos \omega(t - \mu t_d)}{\mu \alpha} \right] \quad (\mu t_d < t \leq t_d) \quad (6.8b)$$

where

$v(t)$ = the displacement of the SDOF relative to the ground

$\dot{v}(t)$ = the velocity of the SDOF relative to the ground

Displacement:

$$v(t) = \frac{A_g}{\omega^2 t_d} \left[\frac{\sin \omega t}{\mu \omega} - \frac{t}{\mu} \right] \quad (0 \leq t \leq \mu t_d) \quad (6.8c)$$

$$v(t) = \frac{A_g}{\omega^2 t_d} \left[\frac{\mu \omega(t - t_d) + \alpha \sin \omega t - \sin \omega(t - \mu t_d)}{\mu \alpha \omega} \right] \quad (\mu t_d < t \leq t_d) \quad (6.8d)$$

At time $t = t_d$, the velocity and displacement are as follows:

$$\dot{v}(t_d) = \frac{A_g}{\omega^2 t_d} \left[\frac{\mu + \alpha \cos \omega t_d - \cos \alpha \omega t_d}{\mu \alpha} \right] \quad (6.9a)$$

$$v(t_d) = \frac{A_g}{\omega^3 t_d} \left[\frac{\alpha \sin \omega t_d - \sin \alpha \omega t_d}{\mu \alpha \omega} \right] \quad (6.9b)$$

A closed-form solution for a symmetrical triangular load (i.e. $\mu = 0.5$) was derived by Chopra (1995b). The displacement was presented by normalizing it with respect to the static displacement. By substituting $\mu = 0.5$ into Equation 6.9 and dividing the result by the static displacement of $-A_g T^2 / 4\pi^2$, it can be shown that Equation 6.9 is degenerated to the solution of a symmetrical triangular load as obtained by Chopra (1995b).

Application of Correction Factors

Using the impulse-momentum transfer as discussed (Equation 6.4), the approximate velocity and displacement can be derived for an SDOF system, which is subjected to a general triangular load (Figure 6.3). At time $t = t_d$, they can be expressed as follows:

$$\dot{v}_e(t_d) = -0.5 A_g t_d \quad (6.10a)$$

$$v_e(t_d) = -\frac{1}{6} [A_g t_d^2 (2 - \mu)] \quad (6.10b)$$

However, as described the approximate velocity and displacements are overestimated due to the negligence of two terms in Equation 6.2. The overestimated velocity and displacement at time $t = t_d$ can be improved by applying correction factors. The correction factors C_1 and C_2 , which are the ratios between the exact response (Equation 6.9) and the approximate response at time $t = t_d$ (Equation 6.10). For this, the angular frequency ω has been replaced by $2\pi/T$, where T is the fundamental period of the SDOF system.

Figure 6.3 - General triangular ground acceleration

Figure 6.4 - Illustration of closed-form solution for SDOF system and one triangular EIGM

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

The correction factors are shown below:

$$C_1 = \frac{\dot{v}(t_d)}{\dot{v}_e(t_d)} = \frac{[\cos 2\pi\alpha(t_d/T) - \alpha \cos 2\pi(t_d/T) - \mu]}{2\pi^2(t_d/T)^2 \mu \alpha} \quad (6.11a)$$

$$C_2 = \frac{v(t_d)}{v_e(t_d)} = \frac{3[\sin 2\pi\alpha(t_d/T) - \alpha \sin 2\pi(t_d/T)]}{4\pi^3(t_d/T)^3 \mu \alpha(2 - \mu)} \quad (6.11b)$$

Thus, the corrected velocity and displacement at an instant t take the form of:

$$\dot{v}_{ec}(t) = \dot{v}_e(t) \times \dot{v}(t_d) / \dot{v}_e(t_d) = C_1 \times \dot{v}_e(t) = C_1 \dot{v}_g(t) \quad (6.12a)$$

$$v_{ec}(t) = v_e(t) \times v(t_d) / v_e(t_d) = C_2 \times v_e(t) = C_2 \times v_g(t) \quad (6.12b)$$

where

$C_1 =$	correction factor for velocity
$C_2 =$	correction factor for displacement
$\dot{v}_{ec}(t) =$	the corrected velocity
$v_{ec}(t) =$	the corrected displacement

Figure 6.5 shows the correction factors for different ratios of t_d/T and three triangular loads ($\mu = 0.1$, $\mu = 0.5$ and $\mu = 0.9$). Generally, the correction factors for both the velocity and displacement tend to 1.0 when t_d/T tends to zero. This means that the approximate solution becomes more accurate as the loading duration of triangular loads decreases. For the same t_d/T value, a right-skewed triangular load (e.g. $\mu = 0.9$) tends to have a correction factor for displacement that are larger than a left-skewed triangular load (e.g. $\mu = 0.1$). This implies that the approximate displacement at time $t = t_d$ under a right-skewed triangular load is more accurate than that of a left-skewed triangular load. Regions of t_d/T values that lead to negative correction factors indicate a phase difference between the estimated and the exact responses. Notably, for an SDOF system subjected to EIGM, the t_d/T value would be expected to be much less than 0.5.

Figure 6.5 - Displacement and velocity correction factors

Under the left-skewed triangular load ($\mu = 0.2$), the estimated velocity and displacement responses, with and without the correction factor, are compared with the exact response for $t_d/T = 0.2$ (Figure 6.6). The approximate response results with correction factors are more similar to the exact response than those without correction factors. The velocity and displacement at the end of Phase I are the initial conditions for response during Phase II. Thus, the approximate velocity and displacement during Phase I should be improved using the correction factors. For an SDOF system subjected to a triangular load, the correction factors are derived and shown in Equation 6.11.

Figure 6.6 - Velocity and displacement response

Initial Conditions for Phase II

As can be seen, there are slight differences between the response at the end of Phase I (time $t = t_d$) when comparing: (a) the proposed approximate method and (b) the closed-form solutions. Further to this, the impact of this difference on Phase II response is studied.

It was discussed that a left-skewed triangular EIGM load produces the most significant difference in the displacement for the response at $t = t_d$ between the approximate method and the closed-form solution (Figure 6.5). Thus, a left-skewed triangular load shall be used for this study. Two methods are used for determining

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Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

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the Phase II response to study the effect of approximate methods in determining the response during Phase II, which are:

- Approximate method is used to determine the corrected response estimates $\dot{v}_{ec}(t_d)$ and $v_{ec}(t_d)$. This is followed by using the approximate response at $t = t_d$ as the initial conditions to determine the response during Phase II using the closed-form solution. In short, the response during Phase I is determined by approximate solutions and Phase II is determined by the closed-form solution. This method shall be called A-C.
- Closed-form solution is used to determine the response for both Phase I and Phase II. This method is termed C-C.

The SDOF system is that shown in Figure 6.2. The responses determined by both A-C and C-C are shown in Figure 6.7.

As can be seen, the magnitude of the response during Phase II is similar for both A-C and C-C. However, there is a time lag between the responses determined by A-C and C-C. In terms of engineering design of structures, such a time lag would not matter much.

Conclusion

The approximate method produces the response at the end of Phase I, which can effectively serve as the initial conditions for Phase II response analysis. By this, the response analysis for Phase I and Phase II can be performed separately which would save computational effort.

Figure 6.7 - Effectiveness of using the approximate response at time $t = t_d$ as the initial conditions for Phase II response

6.1.2 PROPOSED APPROXIMATE METHOD: MDOF SYSTEM

For an undamped, lumped-mass SDOF system, subjected to a triangular load $\mu = (0,1)$ as shown in Figure 6.3, the displacement response is shown in Equation 10b. For a MDOF system with n degrees-of-freedom, the exact modal displacement response can be derived by modal superposition of the response of each mode. Expressing the modal angular frequency of the structure in terms of the modal period of the structure, the exact modal displacement at time $t = t_d$ can be determined.

$$Y_n(t_d) = \frac{\Gamma_n A_g t_d^2 [\alpha \sin 2\pi(t_d / T_n) - \sin \alpha 2\pi(t_d / T_n)]}{8\pi^3 (t_d / T_n)^3 \mu \alpha} \quad (6.13a)$$

$$\Gamma_n = \frac{\boldsymbol{\varphi}_n^T \mathbf{m} \mathbf{1}}{M_n} \quad (6.13b)$$

where

$Y_n(t_d) =$	modal response for nth mode at time $t = t_d$
$\Gamma_n =$	modal participation factor for the n^{th} mode
$T_n =$	modal period of the structure
$\boldsymbol{\varphi}_n =$	mode shape for the n^{th} mode
$\mathbf{m} =$	mass matrix for the MDOF system
$M_n =$	modal mass for the n^{th} mode

Similarly, the estimated modal displacement is as follows:

$$Y_{ne}(t_d) = -\frac{\Gamma_n A_g t_d^2 (2 - \mu)}{6} \quad (6.14)$$

where:

$$Y_{ne}(t_d) = \text{estimated modal response for nth mode at time } t = t_d$$

Then, the correction factor for modal displacement can be shown to take the form:

$$C_{2n} = \frac{Y_n(t_d)}{Y_{ne}(t_d)} = \frac{3[\sin 2\pi\alpha(t_d/T_n) - \alpha \sin 2\pi(t_d/T_n)]}{4\pi^3(t_d/T_n)^3 \mu\alpha(2 - \mu)} \quad (6.15)$$

To approximate the response of an MDOF system to a triangular load, the modal displacement is approximated and corrected using Equations 6.14 and 6.15, respectively. Through the modal combination method, the approximate velocity and displacement responses can be determined for each degree of freedom.

6.1.3 MDOF SYSTEM UNDER EIGM

For the case where an EIGM load is idealized into a series of triangular loads, the approach outlined above is repeated for each triangular load. Figure 6.8 shows the idealized form of the simulated horizontal component of an EIGM arising from detonation a 100-tonne TNT, with a loading density of 10 kg/m³ (Wu et al, 2002).

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

This simulated horizontal component of the EIGM is induced at a point which is 10 m away from the point directly above the detonation of the explosive. This simulated EIGM has a peak ground acceleration (PGA) of about 14 g. At the end of the loading duration $t = 0.5$ s, the time history of the cumulative impulse of this EIGM reaches a zero value and fluctuates slightly about this value thereafter. A numerical example of a two-storey RC frame is analyzed, and the approximate response compared with the exact response using the idealized EIGM. The two-storey frame analyzed in Dhakal and Pan (2003) is used as the MDOF system. The layout and details of the 2-storey RC building frame used for the dynamic response analysis are shown in Figure 6.9. This portal frame supports a side of a 150 mm thick slab ($5 \text{ m} \times 5 \text{ m}$) rested on the beam at each floor. The beams and columns have $230 \text{ mm} \times 300 \text{ mm}$ cross-section reinforced with six longitudinal steel bars of 19 mm diameter (reinforcement ratio $\sim 2\%$) and 10 mm diameter double-legged lateral ties at 100 mm spacing. The mechanical properties of concrete are: compressive strength = 30 MPa; tensile strength = 2 MPa; Poisson ratio = 0.2; strain at peak = $2400 \mu\epsilon$; and initial stiffness = 24.8 GPa. Similarly, the steel properties are adopted as follows: yield strength = 410 MPa; ultimate strength = 615 MPa; breaking strain = 5%; and Young's modulus = 200 GPa.

For the MDOF system, Timoshenko beams are used and only translational masses are considered. While the global horizontal vibration modes had frequencies of 3 Hz and 9 Hz, the local vibrations of the columns had frequencies ranging from 65 Hz to 700 Hz. Thus, the ratio of the duration of the EIGM load to the modal periods t_d/T_n ranged from 0.07 to 5.5. Due to the much smaller modal contribution of the higher frequency local modes with t_d/T_n of 5.5, the error induced by these modes are not significant.

Figure 6.8 - RC frame and EIGM used

The time histories for approximate displacement are compared with those from the exact displacement by closed-form solution (Figure 6.9). As can be seen, the approximate method gives response results that are fairly accurate. While the displacement at the roof is slightly higher than the mid-height of the first storey, both displacements are very small. Further, the displacements at the mid-height are of a higher frequency than that at the storey levels.

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

Based on the approximate displacement response histories, the shear and moment at the base of the column can be determined (Figure 6.10). The shear forces and bending moment are observed to be the greatest at the base of the column. At the base of the column, the maximum absolute values for induced shear force and induced bending moment are 40 kN and 20 kNm, respectively. As the cracking moment of the column is about 80 kNm, it can be seen that the response during Phase I is elastic. The diagonal shear capacity of concrete being 171 kN, and so the induced shear force of 40 kN will not lead to diagonal shear failure. The absolute accelerations determined show that they are the highest at the mid-height of the column.

Figure 6.9 - Approximate and exact deformations

Figure 6.10 - Induced shear and moment

6.1.4 PROPOSED APPROXIMATE METHOD: CONCLUSIONS

Explosion-induced ground motions (EIGMs), which are short duration, are formed by a series of triangular-shaped loads. Taking advantage of the unique characteristics of short-duration EIGMs, an approximate method was proposed to estimate structural response to EIGM.

For EIGM idealized as a series of triangular loads, the displacement and velocity during Phase I are derived from the impulse-momentum relationships. As the displacement and velocity determined by the impulse-momentum relationships are overestimated, a correction factor is proposed. The approximate method have been developed for both SDOF and MDOF systems when subjected to an EIGM. The approximate response has been shown, via two examples of SDOF and MDOF systems, to be close to the exact solutions. Thus, the approximate method can be used effectively to determine the structural response during Phase I. Furthermore, the approximate displacement and velocity at the end of Phase I ($t = t_d$) can be used as the initial conditions for the response analysis during Phase II.

The proposed approximate method has the following characteristics:

- The approximate method can be used to determine the response at the end of Phase I. These response values can be used as the initial conditions for the response analysis of Phase II. Thus, it allows for Phase I and Phase II response analyses to be separated. With this, the model for Phase II response can have fewer degrees of freedom since the response does not exhibit local mode response during this phase.
- Besides the response at the end of Phase I, the method produces a time history of approximate response during Phase I. This is important as it has been observed that the shear force on a column when subjected to an EIGM reaches its peak during the loading duration ($t/t_d \leq 1$). Thus an approximate method should provide a time history of the induced shear force so that *toughness test* for Phase I can be carried out.

- The proposed approximate method has been extended to an MDOF system. This is important as the significant contribution from local modes of a structural element can thus be dealt with via the approximate response.

With this, the study of structural response characteristics and structural damage modeling under EIGMs can be performed with greater ease and speed. Here, the approximate method that has been developed and implemented for an MDOF system subjected to an EIGM will be used.

6.2 PROPOSED DAMAGE ASSESSMENT FOR PHASE I

In this section, a damage assessment for Phase I shall be proposed for a *single column* subjected to EIGM. The proposed damage assessment shall include the following characteristics:

- Damage assessment which is strength-based and also response-based
The choice of using a strength-based damage assessment technique is based on the literature reviews which cited the inappropriateness of using response-based damage assessment for structures subjected to EIGM during Phase I (Ma et al, 2002; Pan et al, 2002). However, it is also required to be response-based since it needs to include the strength-enhancement due to strain rate effects during the response.
- Strength-enhancement effect of high strain rates.
During Phase I, the structural response induced by EIGM exhibits large shear forces at the column base. Thus, this may be critical during Phase I and may be the primary cause of structural failure. However, the shear strength capacity should take into account the strength-enhancement effect of high strain rates.
- Direct shear strength capacity used: Malvar and Crawford (1998b).
- Diagonal shear strength capacity used: Priestley et al (1994).
- Although the induced shear force expected during Phase I is large and would probably lead to direct shear failure. A check shall be made to determine if

the column will fail in direct shear or diagonal shear, which is based on the experimental observations made by Li and Jones (2000).

- The type of failure mode should be predicted such that retrofits can be made to enhance structural resistance to avoid that failure mode.
- As the proposed damage assessment method is designed to be a *toughness test* of a structure during Phase I, it will be based on failure mechanisms. As long as one mode of failure is experienced, the structure is considered to have failed. Here, the modes of failure considered are flexural failure, diagonal shear failure and direct shear failure. Within each failure mode, failure is based on the exceedance of strength capacity. For instance, when the induced shear force exceeds the shear strength capacity, a shear failure mode is deemed to have occurred.

6.2.1 PROPOSED DAMAGE ASSESSMENT PROCEDURE

The procedures of the proposed damage assessment for Phase I are as follows:

- Step 1: Perform the response analysis of Phase I when a structure is subjected to an EIGM. Determine the time histories for induced forces such as (a) induced shear forces and (b) induced bending moment. With that, determine the time histories of the strain rates for the concrete and steel reinforcement. For the concrete and steel reinforcement, the strain rates are the rate of change of strains per unit time. The strains are determined on the assumption that the strains vary linearly along the depth of the composite section. Then, the corresponding dynamic increase factors (DIF) of concrete compressive strength and steel tensile strength are determined. For the DIF of concrete compressive strength, use the formulation by CEB (1993) as shown in Equation 3.8. For the DIF of steel reinforcement tensile strength, use the formulation by Malvar and Ross (1998b) as shown in Equation 3.11.
- Step 2: Compute the time histories of the diagonal shear strength capacity and direct shear strength capacity.

Chapter 6: Proposed Methods and Techniques – Phase I of EIGMDiagonal shear strength capacity

The formulation by Priestley et al (1994) shall be used. The formulation is shown in Equation 4.1 and replicated here in Equation 6.16. The modification made here is to account for some degree of enhancement in the concrete compressive strength as a result of lower strain rate effects. Thus, the static concrete compressive strength as used by Priestley et al (1994) has been replaced by a dynamic concrete compressive strength, f'_{dc} .

$$V = V_c + V_p + V_s \quad (6.16a)$$

$$V_c = 0.29\sqrt{f'_{dc}}(0.8A_g) \quad (6.16b)$$

$$V_p = \frac{h-c}{2a}P \quad (6.16c)$$

$$V_s = \frac{A_v f_{yh} D'}{s} \cot 30^\circ \quad (6.16d)$$

where

V_c = concrete contribution,

V_p = axial load contribution

V_s = truss-mechanism component,

A_v = the total transverse reinforcement

s = spacing along the longitudinal axis of the member

D' = distance between the centers of the peripheral hoop or spiral

f_{yh} = the yield strength of the transverse steel reinforcement

f'_{dc} = concrete *dynamic* compressive strength

A_g = gross cross-sectional area of member

h = overall depth of section

c = depth of the compressive zone of the section

a = for a cantilever column, it is the member length for a column
in reversed bending, it is half the member length

Direct shear strength capacity

The direct shear strength capacity used is that developed by Malvar and Crawford (1998b), which is as follows:

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

$$v_{du} = 0.14f'_c + \frac{1.2}{d}(N_c + A_{vf}f_{dy}) \leq 0.6f'_{dc} \quad (6.17)$$

where

N_c = applied axial compression force per unit width (N/mm)

A_{vf} = area of shear friction reinforcement per unit width (mm^2/mm)

d = distance from the extreme compression fiber to the centroid of the tension reinforcement (mm)

f'_{dc} = concrete dynamic ultimate compressive stress (MPa)

f_{dy} = reinforcement dynamic yield stress (MPa)

Step 3: Predict the type of the shear characteristic that would arise based on the experimental observations by Li and Jones (2000) as shown in Equation 3.13. For this purpose, the induced shear force and induced bending moment are required. Here, Equation 3.13 is modified based on a dimensionless parameter as follows:

$$\text{Direct shear: } \frac{M}{QH} \leq 0.433 \quad (6.18a)$$

$$\text{Diagonal shear: } \frac{M}{QH} > 0.433 \quad (6.18b)$$

where

Q = induced shear force

M = induced bending moment

H = thickness of the section in the direction of the ground motion

Depending on the ratio of $M/(QH)$ in Equation 6.18, the direct shear strength capacity of Equation 6.17 shall be used or the diagonal shear capacity of Equation 6.18 shall be used for failure determination.

Step 4: For the shear force alone during Phase I, a strength-based damage index is proposed. The proposed damage index is designed for a toughness test. Thus no attempts shall be made to add the independent contributions to the damage from the effects of shear force and bending moment. That is, the

damage index shall be taken as the maximum of two ratios, each of which is the ratio of the induced force or bending moment to the respective strength capacity. For an instant in time t , the proposed damage index for Phase I is as follows:

$$\text{Damage Index} = \text{Max} \left(\frac{Q}{Q_c}, \frac{M}{M_c} \right) \quad (6.19)$$

where

Q = induced shear force as determined in Step 1

Q_c = shear strength capacity as outlined Step 3

M = induced bending moment as determined in Step 2

M_c = currently taken as the yielding moment capacity

$\frac{Q}{Q_c}$ = shear damage index

$\frac{M}{M_c}$ = flexural damage index

Note: Failure when Damage Index > 1.0

A flowchart describing the procedures of the proposed damage assessment is shown in Figure 6.11.

Application Examples

First, the proposed damage assessment technique is applied to a single column subjected to a triangular load. Later, the damage assessment of this single column is compared with the damage assessment of a column within a multi-storey RC frame. If the damage assessment of a single column compares well with that of the same column within an RC frame, this would provide a simplified damage assessment of an RC frame subjected EIGM during Phase I. The computational effort incurred by a single column is less and would provide an insight into the induced structural damage by an EIGM during Phase I.

Figure 6.11 - Flowchart for proposed damage assessment

6.2.2 DAMAGE ASSESSMENT: FIXED-FIXED COLUMN

In Chapter 5, it was shown that the induced shear force and bending moment for a column subjected to an EIGM load is the largest at the column base ($y = 0.0h$). This is regardless of the boundary conditions present in the column. Thus, the damage assessment shall be demonstrated using a fixed-fixed column subjected to an EIGM. The column is as shown in Figure 5.1. This column is to represent the column at the first storey level of a 2-storey RC frame (Figure 6.8). The RC-frame was used by Pan et al (2004b). In Section 5.2, the response analysis is performed for a fixed-fixed column subjected to a triangular EIGM load (Figure 5.1).

It is shown that EBT-FEM modeling produces results similar to TBT-FEM model of the column. This section continues with application of the proposed damage assessment on the fixed-fixed column that is subjected to the same triangular EIGM load (Figure 5.1). The response analysis for the damage assessment shall be based on that determined by the TBT-19 model of the fixed-fixed column in Section 5.2. As described in Step 1 of the procedures (Figure 6.11), determination of the induced shear force and bending moment has been performed in Section 5.2 and shown in Figure 5.4.

Strain Rates and Shear Strength Capacity

This stage of the analysis follows Steps 1 to 2 of the procedures of the proposed damage assessment. The strain rate and DIF results are shown for the column base where the induced shear force is the largest (Figure 6.12). The observations are as follows:

- The strain rates that are determined for concrete and steel rise to their peak values of about 1 s^{-1} at the middle of Phase I (Figure 6.12a). Later, they decrease to a magnitude of about -1.0 s^{-1} . At time $t > t_d$, the strain rates are negative, which represents unloading of the concrete.
- As a result of the high strain rates, the DIF for concrete and steel is about 1.4 (Figure 6.12b). This means that there is about a 40% enhancement in the compressive strength of concrete and similarly in the tensile strength of steel

reinforcement. At time $t > t_d$, the unloading of concrete results in a DIF is 1.0, which means that the compressive strength of concrete is equivalent to the static strength.

- In the $M/(QH)$ time history, it is seen to increase with time (Figure 6.12c). This means that the induced moment is increasing relative to the induced shear force. Such a trend can be explained as follows. When a column is subjected to EIGM, the shear waves propagate along the structure. This would lead to the inertia forces of the column being *activated* as the wave propagates from the column base towards the free end. This inertia force then drives the structure into vibration. At this point, the large inertia force induces a large shear force in the column. As such, this process takes time and there is a slight time lag before the column shows significant deformation. It is when the column shows significant deformation that the column begins to bend significantly with induced moment. In short, the induced shear forces are higher than the induced moment at the onset of the column response to the EIGM. This implies that the structure is under a state of direct shear at the onset and within the early stages after the EIGM begins. As the column starts to experience significant deformation, the induced moment increases with the increased deformation and thus $M/(QH)$ increases later.
- Closer inspection of the time history of $M/(QH)$ shows that it crosses the value of 0.433 at about $t/t_d = 0.15$. This is used to determine the type of induced shear force at the column base during the response. By Equation 6.14, direct shear is induced at the column base at the initial stages of column response ($t/t_d \leq 0.15$). After this ($t/t_d > 0.15$), the induced shear force at the column base is diagonal shear as $M/(QH)$ exceeds 0.433 (Equation 6.18).
- With the type of induced shear determined during the response, the time history of the shear strength capacity is determined (Figure 6.12d). At the initial stage of the column response ($t/t_d \leq 0.15$), the direct shear strength capacity is used and increases from 250 kN to 280 kN as a result of the increasing DIF of concrete compressive strength and steel reinforcement

tensile strength. Later, the diagonal shear strength capacity is used and is about a value of 175 kN, which is slightly higher than the diagonal shear strength capacity of 171 kN. At time $t > t_d$, the concrete is unloading leading to a diagonal shear strength of 171 kN, which is equivalent to the diagonal shear strength capacity under static condition. It can be seen that under dynamic loading, the direct shear strength is about twice that of diagonal shear strength mentioned.

Damage Assessment

This is the final step of the proposed procedures for damage assessment. Figure 6.13 shows the time histories for the shear damage index and flexural damage index. The observations are as follows:

- The time history of the shear damage index (Q/Q_c) shows that it increases with time to a peak of about 0.55 at time $t/t_d > 0.7$. There is an abrupt increase of the shear damage index from 0.5 to 0.55 at time $t/t_d > 0.7$. This is due to the decrease of the shear strength capacity to the diagonal shear strength under static conditions, as a result of the unloading. After this, it decreases to a value of about 0.3. Thus, the peak contribution of the induced shear force to the damage index is at time $t/t_d = 0.7$, when the induced shear force is the largest.
- The time history of the flexural damage index (M/M_c) shows that it increases to a peak of about 0.4 at the end of the loading duration ($t/t_d = 1$). The trend observed in this time history is because of the increasing column displacement to a peak value at the end of the loading duration ($t/t_d = 1$). As the column bending moment is induced by the column displacement, a similar trend can be seen in this term.
- At a given time, the damage index is the maximum of the shear damage and the flexural damage. The increase in the early part of the damage index indicates increasing shear force induced. The index decreases between time $t = 0.7t_d$ and time $t = 0.9t_d$. After this, it increases again towards the end of the loading duration ($t/t_d = 1$) due to the increased bending moment.

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

- The value of the proposed damage index does not exceed 1.0, which indicates no failure by either shear failure or flexural failure, during Phase I ($t/t_d > 1$). However, moderate damage levels may be experienced by the column. The early part of the damage index (0.0 to $0.9t_d$) is related to shear damage, while the last section ($0.9 t_d$ to $1.0t_d$) is related to flexural damage.

(a) Concrete and steel strain rate time histories

(b) DIF time histories for concrete compressive strength and steel tensile strength

(c) $M/(QH)$ time history

(d) Shear strength capacity time history

Figure 6.12 - Response of fixed-fixed column including strength-enhancement effect using TBT-19 model

Figure 6.13 - Proposed damage index for fixed-fixed column during Phase I

6.2.3 CASE STUDY: DIAGONAL SHEAR FAILURE

This section presents the damage assessment of the fixed-fixed column subjected to a scaled EIGM. The objective is to present a case where shear failure would be experienced by the fixed-fixed column when subjected to a more damaging triangular EIGM load. The type of shear failure featured here is the diagonal shear failure. The scaled EIGM is produced by changing the characteristics of the EIGMs. TBT-19 model of the fixed-fixed column shall be used. Earlier, the triangular EIGM used the following characteristics:

- Peak ground velocity or peak particle velocity (PPV) of 1 m/s
- Peak ground acceleration (PGA) of 80 g
- Duration, $t_d = 2.5$ ms
- Principal frequency of 200 Hz

In this section, the triangular EIGM load is scaled to have the following characteristics:

- Peak ground velocity or peak particle velocity (PPV) of 2 m/s
- Peak ground acceleration (PGA) of 160 g
- Duration, $t_d = 2.5$ ms
- Principal frequency of 200 Hz

As can be seen, only the PPV is scaled by a factor of two with the other parameters kept the same. Thus, the amount of input impulse of the ground motion is doubled but the input rate of ground motion impulse is kept the same.

Response results

Figure 6.14 shows the results of the time histories for the induced shear force and bending moment at the column base. The observations are as follows:

- The induced shear force reaches a peak value of about 185 kN while the induced bending moment reaches a peak value of about 90 kNm. While the induced shear force reaches its peak at about time $t/t_d = 0.7$, the induced bending moment reaches its peak value at the end of the loading duration ($t/t_d = 1.0$). This is similar to the trend observed with the EIGM of half the PPV value. The reason for the induced shear force reaching its peak value prior to the induced bending moment reaching its peak value is due to the large inertia forces.
- As compared to the EIGM of half the PPV value (Figure 5.4), both the shear force and bending moment induced by the EIGM here are larger. This can be explained by the inherent difference in the EIGMs used in both instances. While the EIGM used had the same principal frequency as the one in Section 5.2, it has a PPV that is double of that used in Section 5.2. The impulse transmitted to the column is thus double and within the same duration as compared to the EIGM in Section 5.2. This means that the rate of transfer of impulse from the EIGM to the column is also doubled. This increased rate of transfer of impulse causes the increased shear force and bending moment induced for the same fixed-fixed column.

Damage Assessment

The proposed damage assessment process is undertaken here. Figure 6.15 shows the time histories required for Steps 2 to 3 of the procedures for damage assessment. The following comparisons can be made between the results from this scaled EIGM and the EIGM with half the PPV value:

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

- Here, the concrete strain rates are higher as compared to an EIGM with half the PPV value. This is due to the increased rate of transfer of impulse from the ground to the column. However, similar to the EIGM with half the PPV value, there is also unloading of the concrete at time $t/t_d > 0.7$.
- As a result, the DIF for concrete compressive strength and steel tensile strength are also slightly higher as compared to the EIGM with half the PPV value. Due to the unloading of the concrete at time $t/t_d > 0.7$, the DIF for concrete compressive strength and steel tensile strength are equal to 1.0 at time $t/t_d > 0.7$.
- Notably, the type of shear predicted here also differs from that predicted in Section 5.2. Here, the transition from direct shear behavior to diagonal shear behavior takes place at time $t/t_d = 0.2$. For the EIGM used with half of the PPV value, this transition takes place at an earlier time $t/t_d = 0.1$. Furthermore, due to the unloading at time $t/t_d > 0.7$, the shear strength capacity is equal to the diagonal strength capacity under static conditions.

Figure 6.16 shows the damage assessment of the fixed-fixed column. The following are the observations made:

- The shear damage index is larger from the induced shear force than it is for the induced bending moment. Such is also the case for the same column subjected to the EIGM used in Section 5.2
- The shear damage index is larger than one which indicates that there will be diagonal shear failure. This is due to the reduced shear strength as a result of the unloading of concrete at time $t/t_d > 0.7$.

Figure 6.14 - Induced shear force and bending moment for scaled EIGM

Figure 6.15 – Response of fixed-fixed column to scaled EIMG including strength enhancement effect

Figure 6.16 - Damage assessment for column under scaled EIGM

6.2.4 CASE STUDY: DIRECT SHEAR FAILURE

As shown by the scaling of the EIGM, the mode of shear failure that is observed so far is diagonal shear failure. This section shows a case where direct shear failure is experienced. This is shown by using the same fixed-fixed column subjected to another scaled EIGM. This scaled EIGM has the following characteristics:

- Peak ground velocity or peak particle velocity (PPV) of 3.5 m/s
- Peak ground acceleration (PGA) of 1,120 g
- Duration, $t_d = 0.625$ ms
- Principal frequency of 800 Hz

Damage assessment

The damage assessment technique is applied to determine the type of shear failure. Figure 6.17 shows the application of the proposed damage assessment for this case. From the time history of $M/(QH)$, it can be seen that the transition from direct shear behavior to diagonal shear behavior occurs at time $t/t_d = 0.6$. From Figure 6.17b, the shear damage has reached 1.0 and beyond. With the transition of shear behavior known, this indicates a direct shear failure at time $t/t_d = 0.5$. As such, the shear damage and flexural damage beyond time $t/t_d = 0.5$ is not shown.

As compared to the EIGM that caused diagonal shear failure, the EIGM that caused direct shear failure has a higher PPV and principal frequency.

Figure 6.17 - Damage assessment of direct shear failure case

6.2.5 DISCUSSIONS

The proposed method for damage assessment is tested for a fixed-fixed column and has been shown to have the following characteristics:

- The strength has been shown to increase by about 40% and so the strength-enhancement effect of strain rate effects is significant. The proposed damage assessment method includes this strength-enhancement effect of strain rate effects.
- The proposed method for damage assessment is applicable as a *toughness test* for structures during Phase I ($t/t_d = 1$). It regards the structure has failed when either a flexural failure or shear failure is experienced. The failure in both cases is based on exceedance of a strength capacity. Within the proposed damage assessment method, a feature to determine the type of

shear behavior is present. This allows for a more accurate prediction of the shear strength capacity and also the type of shear failure mode. With this, section designs can be tested against EIGM loads and the possible failure modes established. This can lead to changing the sectional designs in view of the possible failure mode.

- Shear failure of a member subjected to EIGM may be in the form of diagonal shear failure or direct shear failure. For direct shear failure to occur, the EIGM would have to possess a higher PPV and/or principal frequency as compared to one that would lead to diagonal shear failure.
- The strength-based damage assessment that was defined by Aoyama (1981) is not applicable for the damage assessment of structures subjected to EIGM. This is because it is a strength-based method that does not include the strength-enhancement due to strain rate effects. For the inclusion of the strength-enhancement due to strain rate effects, a response time history of the strain rates is required. Here, the proposed damage assessment is based both on strength and structural response.

6.3 PROPOSED SIMPLIFIED SINGLE COLUMN ANALYSIS

The response analysis for Phase I requires great computational effort. In the case where the structure is a multi-storey RC frame, the computational effort can be large. Furthermore, if material-level modeling is used for the analysis, there is greater computational effort. Here, a simplified analysis is proposed for Phase I. It is proposed that the investigation of shear failure for a multi-storey RC frame during Phase I can be simplified by performing the shear failure analysis on the base column of that storey level.

The proposed simplified analysis can be summarized as described below. It has been shown that the shear forces and bending moments induced are largest at the base of a column. Thus, for a multi-storey RC frame, the critical locations during Phase I will be the first-storey columns. As it has been demonstrated that the boundary conditions of a column does not significantly affect the shear forces and bending moment determined at the column base, a simplified method for determining the critical shear at the base of a column can be performed using a fixed-fixed column. This can provide a simplified analysis for the *toughness test* of a multi-storey RC frame during the loading duration of an EIGM. In the simplified analysis, the damage assessment method proposed in Section 6.2 is used to highlight the possible failure mechanisms.

To demonstrate its feasibility, the results of the simplified method are compared with those of the analysis of a multi-storey RC frame subjected to an EIGM (Pan et al, 2004b).

6.3.1 SCALING OF EIGMs

Short of having a spectrum of EIGMs to work with, scaling of EIGMs has been applied to create a family of EIGMs from an original EIGM (Pan et al, 2004b). This produces many EIGMs with varying PGA, PPV and principal frequency. To do this, scaling multipliers are applied to the characteristics of PPV and principal frequency.

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

For instance, when a PPV multiplier of 2 and a frequency multiplier of 1 are applied to the original EIGM, the scaled EIGM will have twice the PPV of the original EIGM and the same principal frequency of the original EIGM. This is the case that is used for Section 6.2.3. The original EIGM has a PPV of 1 m/s and a principal frequency of 200 Hz. By scaling, the scaled EIGM has a PPV of 2 m/s and a principal frequency of 200 Hz.

As a further example, a case where a PPV multiplier of 3.5 and a frequency multiplier of 4 are applied to the original EIGM is described. Here, the scaled EIGM would have a PPV 3.5 times that of the original EIGM and a principal frequency 4 times of the original EIGM. This is the case of the scaled EIGM used in Section 6.2.4. The original EIGM had a PPV of 1 m/s and a principal frequency of 200 Hz. When the scaling multipliers were applied, the scaled EIGM had a PPV of 3.5 m/s and a principal frequency of 800 Hz.

Specific to a triangular EIGM loads, a relationship exists among the PPV, the PGA and the principal frequency of the EIGM. The derivation starts at the PPV being the integral of the acceleration time history with respect to time (Equation 6.20a). Figure 6.18 shows the acceleration and velocity time histories of a single triangular EIGM load. For a single triangular load that occurs in a duration of t_d , the period is $2t_d$ and so the principal frequency is $(0.5/t_d)$. This leads to the relationship that is shown in Equation 6.20b.

$$\text{PPV} = \frac{\text{PGA} \times t_d}{2} = \frac{\text{PGA} \times 2t_d}{4} = \frac{\text{PGA}}{4f_g} \quad (6.20a)$$

$$\text{PPV} \times f_g = 0.25\text{PGA} \quad (6.20b)$$

where

f_g = the principal frequency of an EIGM

When scaling EIGMs, the following multipliers are defined.

$$m_v = \frac{(\text{PPV})_1}{(\text{PPV})_0} \quad (6.21a)$$

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

$$m_a = \frac{(PGA)_1}{(PGA)_0} \quad (6.21b)$$

$$m_f = \frac{(f_g)_1}{(f_g)_0} \quad (6.21c)$$

where

$m_v =$	PPV multiplier for scaling EIGMs
$PPV_1 =$	the peak particle velocity of scaled EIGM
$PPV_0 =$	the peak particle velocity of original EIGM
$m_a =$	PGA multiplier for scaling EIGMs
$PGA_1 =$	the peak ground acceleration of scaled EIGM
$PGA_0 =$	the peak ground acceleration of original EIGM
$m_f =$	frequency multiplier for scaling EIMGs
$f_1 =$	the principal frequency of scaled EIGM
$f_0 =$	the principal frequency of original EIGM

By combining the relationships described in Equations 6.20 and 6.21, Equation 6.22 is deduced.

$$m_a = m_v \times m_f \quad (6.22)$$

Figure 6.18 - Acceleration and velocity time histories for a triangular EIGM load

6.3.2 DAMAGE ASSESSMENT USING MATERIAL-LEVEL MODELING

A damage assessment was performed for an RC frame as shown in Figure 6.9, subjected to scaled EIGMs (Pan et al, 2004b). Material-level modeling was used for the response analysis. The failure mechanisms that were investigated included flexural failure and shear failure. The following describes the failure criteria for the flexural failure and shear failure in Pan et al (2004b):

- In the case of flexural failure, the failure criterion was based on response using material strains.
- The failure criterion for shear failure was strength-based. An exceedance of shear strength capacity would indicate a shear failure. However, the shear strength capacity did not include the strength-enhancement effect due to strain rate effects. Also, direct shear failure was not considered as a possible failure mechanism.

The structure used is a 2-storey RC frame as shown in Figure 6.9a. The ground shock used is a simulated EIGM at 50 m from the source of detonation. The explosion was 250 ton of TNT with a loading density of 10 kg/m³. Figure 6.19 shows the acceleration time history and the acceleration spectra of the simulated EIGM.

Representative Characteristics of an EIGM

For the simulated EIGM, the maximum ground acceleration is 123 g and the minimum ground acceleration is -103 g. Of these two values, a representative PGA is required for the purpose of scaling using Equation 6.22. Likewise, the maximum amplitude for acceleration spectra corresponds with a frequency of about 189 Hz (Figure 6.19). Within the acceleration spectra, there exist other frequency bands. Thus, a representative frequency is required for the purpose of scaling using Equation 6.18. The following describes the procedures used for determining the representative PGA, PPV and principal frequency for the simulated EIGM:

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

- As shown in Figure 6.19, the mean horizontal PGA of the EIGM is 113 g. Similarly, the mean PPV of the horizontal excitations determined from the corresponding velocity time histories is 0.91 m/s.
- The determination of a representative principal frequency of the acceleration time history of the standard EIGM is shown in Figure 6.19. A horizontal line is drawn in the acceleration Fourier spectra at half of the peak spectral acceleration. Vertical lines are drawn from the two extreme intersection points of this horizontal line with the Fourier spectra. The average of the two frequencies determined by these two vertical lines is taken as the principal frequency. If the Fourier amplitudes between these two frequencies are very asymmetric, the frequency at the centroid of the area covered by the Fourier spectra within these two frequencies should be used as the principal frequency. In this case, the arithmetic average of the two frequencies is used for the standard EIGM. The principal frequency of the acceleration record of the standard EIGM thus computed is approximately equal to 185 Hz, as shown in Figure 6.19.

Figure 6.19 - Simulated EIGM at 50 m from explosion (Pan et al, 2004b)

Parametric Study for Damage Assessment

The 2-storey RC frame in Figure 6.9 was subjected to a family of scaled EIGMs (Pan et al, 2004b). Material level modeling was used to in the response analysis of

the 2-storey RC frame subjected to the scaled EIGMs. The results are shown in Figure 6.20. The following observations could be:

- For EIGMs with PPVs that are less than 2 m/s, as the principal frequency of the EIGM increases a transition is seen between the two types of failure as defined by Pan et al (2004). With increasing principal frequencies, the failure mechanism changes from flexural failure (A) to shear failure (C). With increasing principal frequencies, the local mode contribution becomes more significant leading to increased shear forces induced. This causes the transition from flexural failure to shear failure.
- For EIGMs with PPVs larger than 2 m/s, with increasing principal frequencies, the transition in failure modes is also experienced. The transition is made from flexural failure (A) to a combination of shear failure and flexural failure (B) and then to shear failure (C).

However, the damage assessment of Pan et al (2004b) can be improved as follows:

- The EIGMs that lead to a combination of shear failure and flexural failure needs to be defined more clearly. By this, it means that the chronology of failure needs to be established. During the loading duration, it is more probable that the structure would experience one form of failure mechanism before the other. It is unlikely that both failure mechanisms are experienced simultaneously at the same time. Thus, a distinction needs to be established regarding the combination of two failure mechanisms. This would allow retrofits to be made to enhance the resistance of a structure against the failure mechanism that is to be experienced.
- Furthermore, several considerations are not made in this damage assessment which includes: (a) strength-enhancement due to strain rate effect, (b) a distinction between diagonal shear failure and direct shear failure and (c) the possible case of beam-column joints failing.

Nevertheless, the proposed simplified method can be tested against the observations made by Pan et al (2004b) for the feasibility of using a single column for the damage assessment of a multi-storey frame.

Figure 6.20 - Failure mechanisms determined for 2-storey RC frame (Pan et al, 2004b)

6.3.3 FEASIBILITY OF APPLYING THE PROPOSED SIMPLIFIED METHODOLOGY

This section tests the feasibility of performing damage assessment of a multi-storey RC frame when subjected to EIGMs by performing the proposed simplified methodology on a critical column of the frame. The following considerations are made in using the proposed simplified methodology for damage assessment:

- Perform the damage assessment of a multi-storey RC frame through the damage assessment of the column at the first-storey level. This is where the induced shear forces and bending moments are the largest within the RC frame.
- For the single column, a fixed-fixed boundary condition will be used as it has been shown that the boundary condition at the top of the column does not significantly affect the shear forces and bending moments induced at the column base.
- The proposed damage assessment method shall be used, which possesses certain characteristics not considered by Pan et al (2004b) such as (a) strength-enhancement effects and (b) direct shear as a failure mode.

- When a particular failure mode precedes another failure mode, the latter shall not be considered.

Through a similar parametric study using the scaled EIGMs, a fixed-fixed column is subjected to many EIGMs. The possible failure modes for varying PPVs and principal frequencies are presented. The results are compared with those produced earlier for damage assessment by Pan et al (2004b), as a gauge of the applicability of the proposed method. The fixed-fixed column used is as shown in Figure 5.1 which has the same properties as the first-storey column of the 2-storey RC frame (Figure 6.9). The TBT-19 model shall be used (Section 5.2) in the simplified methodology.

Among the many parametric study results for the response of the fixed-fixed column subjected to all the EIGMs, some cases shall be presented to illustrate the possible failure mechanisms.

Case 1: Flexural Failure

An EIGM which led to flexural failure at the base of the fixed-fixed column had the following characteristics: (a) principal frequency of 100 Hz, (b) PPV of 2.7 m/s, (c) t_d of 60 ms and (d) PGA of 183 g.

Figure 6.21 shows the time histories for $M/(QH)$ and the damage terms up to the point of flexural failure ($t/t_d = 0.15$). The following observations can be made:

- The transition from direct shear behavior to diagonal shear behavior took place at time $t/t_d = 0.005$. This is attributed to the lower principal frequency of the scaled EIGM at 100 Hz, which is not large enough to induced significant local mode response. Thus, the induced shear force is not high enough to cause shear failure.
- The flexural damage index reaches 1.0 slightly ahead of the shear damage index. Thus, this case is deemed as a flexural failure case. For the same EIGM, the 2-storey RC frame also exhibits a combination of flexural failure

and shear failure. Given the high damage indices for both flexural and shear, the actual failure may take place earlier with a combination of shear and flexural damages.

Figure 6.21 - Flexural failure of fixed-fixed column under scaled EIGM

Case 2: Diagonal Shear Failure

The EIGM that caused diagonal shear failure of the fixed-fixed column as shown in Figure 6.22, had the following characteristics: (a) principal frequency of 360 Hz, (b) PPV of 1.8 m/s, (c) t_d of 16 ms and (d) PGA of 500 g.

In Figure 6.22 it can be seen that the shear damage index reaches 1 at about time $t/t_d = 0.14$. As the ratio $M/(QH)$ is greater than 0.433 when the shear damage index reaches 1.0, the failure mechanism as determined is diagonal shear failure.

Figure 6.22 – Diagonal shear failure of fixed-fixed column under scaled EIGM

Case 3: Direct Shear Failure

The EIGM that caused direct shear failure of the fixed-fixed column had the following characteristics: (a) principal frequency of 1080 Hz, (b) PPV of 5.5 m/s, (c) t_d of 9 ms and (d) PGA of 4068 g.

Figure 6.23 shows the damage assessment of the column undergoing direct shear failure when subjected to the scaled EIGM. It can be seen that shear damage index reaches 1.0 before the flexural damage index. At this instant, the ratio $M/(QH)$ is less than 0.433 and so the failure mechanism as determined is direct shear failure.

Figure 6.23 - Direct shear failure for fixed-fixed column under scaled EIGM

Damage Assessment of a Fixed-Fixed Column

The parametric study results for the damage assessment of the fixed-fixed column are shown in Figure 6.24b. The results of the damage assessment by Pan et al (2004b) are shown in Figure 6.24a for comparison with the results shown in Figure 6.24b, obtained from the proposed simplified method. The following observations can be made:

- There are EIGMs that do not lead to any failure during the loading duration in terms of the PPV and principal frequencies of the EIGMs.
- The general trends presented in both damage assessment methods show that when the PPV of an EIGM is larger than 2 m/s, as the principal frequency is increased, there will be a transition from flexural failure to shear failure.
- However, a distinction is made between diagonal shear failure and direct shear failure when using the proposed damage assessment method, besides the distinction made between diagonal shear failure and flexural failure.
- For EIGMs of a given PPV, a higher principal frequency is required for direct shear failure as compared to that which caused diagonal shear failure.
- It can be seen that the damage assessment using a fixed-fixed column can produce a similar map of failure modes for a multi-storey RC frame. The lower bound of failure modes, which represents the EIGMs that do not cause failure, is similar by both damage assessment methods. However, the proposed simplified method required less computational effort than that by Pan et al (2004b).

From this, it has been shown that the proposed simplified method produced similar failure maps of a 2-storey RC frame as compared to those produced by Pan et al (2004b). However, the proposed simplified method showed direct shear failure which was not considered by Pan et al (2004b). Furthermore, the proposed simplified method made a distinction between diagonal shear failure and flexural failure when subjected to certain EIGMs. In addition, the proposed simplified method considers strength enhancement due to strain rate effects. Lastly, the proposed simplified method requires less computational effort than that the damage

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

assessment method by Pan et al (2004b). Thus, for RC frames subjected to EIGMs, the proposed simplified method is suitable for assessing the damage during Phase I.

Figure 6.24 - Comparing damage assessment between fixed-fixed column and Pan et al (2004b)

6.4 SUMMARY

The following developments have been made within this chapter:

- An approximate response method was proposed for Phase I. It was demonstrated for an SDOF system and extended to an MDOF system. Since it can be used for an MDOF system, the proposed approximate method can include the local mode contributions, which are significant when structures are subjected to EIGMs. It was shown to provide accurate response analysis for the response during Phase I of structures subjected to EIGM. Also, it was shown that it can be used to attain the responses at the end of Phase I, which serve as the initial conditions for Phase II response. In so doing, the Phase I and Phase II analysis can be separated. Thus, Phase I and Phase II responses do not have to be analyzed in one go. This would reduce much computational effort since a large number of degrees of freedom which are required to incorporate the local mode responses in Phase I, are not required in Phase II response.
- A damage assessment for the shear force in Phase I was proposed for structures subjected to EIGM. This proposed damage assessment includes the strength-enhancement effect of strain rate effects. Furthermore, for a structure subjected to an EIGM, the proposed method allows one to perform a *toughness test* of the structure during Phase I.
- A further simplification was proposed for the damage assessment of multi-storey RC frames when subjected to EIGMs. This method is applicable for the toughness test during Phase I and involves studying the damage on the column at the first-storey level, where large forces are induced during Phase I. The proposed damage index is used and the results were compared with those from the material-level modeling by Pan et al (2004b). The proposed simplified methodology for damage assessment shows results similar to those from the material-level modeling by Pan et al (2004b). Furthermore, the distinction between diagonal shear failure and direct shear failure can be made with the proposed method as opposed to the earlier work (Pan et al, 2004b). The proposed simplification, based on the structural level model,

Chapter 6: Proposed Methods and Techniques – Phase I of EIGM

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has an advantage in that it requires less computational effort as compared to the one based on the material-level modeling.

- During Phase I, the possible modes of shear failure that a column may experience are diagonal shear failure or direct shear failure. For a column, an EIGM that could cause direct shear failure would have to have higher PPV and principal frequency than that causing diagonal shear failure.

7 DAMAGE ASSESSMENT OF A 6-STOREY RC FRAME – PHASE II OF EIGM

In Chapter 6, the proposed approximate method, proposed damage assessment method and proposed simplified method were used for the *toughness test* of a structure during Phase I. If the structure does not fail during Phase I, it would continue the response process into the free vibration phase and may experience large displacements during Phase II. As such, the structure must have sufficient ductility during Phase II to prevent structural failure. This section describes the *ductility test* for Phase II. Particular to this ductility test is the inclusion of a possible joint failure mode during Phase II.

7.1 FRAMEWORK OF DAMAGE ASSESSMENT FOR PHASE II

When frames are subjected to EIGMs, the local modes respond significantly during Phase I. During Phase II, the global modes associated with floor(s) as the main mass points respond more significantly (Lu et al, 2001; Pan et al, 2002; Dhakal and Pan, 2003). Thus for Phase II response analysis, only the global modes need to be included. For Phase II, the response analysis and damage assessment should therefore consider the following:

- Hysteretic post-elastic behavior for members (Park and Ang, 1985). Hence, the response analysis is non-linear.
- Joint failure as defined when inter-storey drift exceeds 3% of the storey height (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003).
- The damage assessment used is a response-based damage assessment technique proposed by Park and Ang (1985).
- In determining the response for Phase II, the initial conditions for the free vibration during Phase II are determined. This is determined by modeling the frame for global modes only, beginning the analysis from Phase I and continuing into Phase II.

For this section, the EIGM used is the simulated one as shown in Figure 6.19.

7.2 FRAME DETAILS

The layout and details of the 6-storey RC building used is shown in Figure 7.1. The concrete shear capacity determined based on the equations by Priestley et al (1994) is 1.6 MPa and the ultimate moment capacities, M_u , is 161 kNm for the exterior columns and 360 kNm for the interior column, respectively.

7.2.1 MODE SHAPES AND MODAL FREQUENCIES

For the nonlinear analysis in Phase II, the analytical software IDARC2D was used (Valles et al, 1996). The frame is modeled for global modes only, where the natural modal frequencies and mode shapes are shown in Figure 7.2. The fundamental frequency is about 1.0 Hz and the second mode shape shows a greater inter-storey drift on the 6th Storey level.

Chapter 7: Damage Assessment of a 6-storey RC Frame – Phase II of EIGM

Figure 7.1 - Elevation and details of a 6-Storey RC frame

Chapter 7: Damage Assessment of a 6-storey RC Frame – Phase II of EIGM

Figure 7.2 - Mode shapes for 6-storey RC frame

7.3 RESPONSE ANALYSIS – GLOBAL MODES

During Phase II, the maximum absolute responses are shown in Table 7.1. The maximum response of lateral displacement occurs in Phase II due to the short duration of the EIGM. The maximum displacement at the top of the frame is around 229 mm, which corresponds to about 0.18 % average inter-storey drift. Figure 7.3a shows the snapshot of the RC building at the absolute maximum responses of displacement and associated bending moments of the columns. From the maximum displacement snapshot, it suggests that building vibrates mainly in the fundamental global mode with less oscillation of the higher-order local vibration modes.

Table 7.1 - Maximum absolute response of frame

Storey	Displacement (mm)	Storey Drift (mm)	Storey Drift Ratio (%)
Roof	229	26	0.8
ST6	218	113	3.2
ST5	131	22	0.6
ST4	110	32	0.9
ST3	80	40	1.1
ST2	40	40	1.1

Figure 7.3- Snapshots at the maximum roof displacement of 229 mm

Figure 7.4 - Lateral displacement time histories at different storeys

Lateral displacement response histories at different storeys are extracted and shown in Figure 7.4. It can be seen that the absolute maximum displacement is attained in Phase II.

Figure 7.5 shows the acceleration response time histories at the different storeys. Unlike the 2-storey frame, this 6-storey frame does not necessarily display the maximum acceleration at all different storeys during Phase I. Instead, the maximum absolute acceleration occurs in Phase II. As local modes were not included in this analysis, the acceleration time histories for the higher modes during Phase I are not shown.

Figure 7.5 - Acceleration time histories at different storeys

Furthermore, Figure 7.6 shows the time histories for the inter-storey drift at the different storeys. The largest inter-storey drift of 113 mm (i.e. 3.2 % drift ratio) occurs at the 6th storey (ST6). In accordance with the experimental observations, the 3.2 % inter-storey drift ratio at ST6 would have resulted in joint failure (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003) for the type of construction studied.

Figure 7.6 - Inter-storey drift time histories at different storeys

As determined based on the damage assessment method by Park and Ang (1985), the global structural damage during Phase I was 0.01 (i.e. moderately damaged) while that during Phase II was 0.09 (i.e. moderately damaged). Table 7.2 shows the breakdown of the column damage indices for each storey during Phase II. The damage indices for Phase I (Park and Ang, 1985) reflect insignificant damage levels. Figure 7.7 shows the time histories of the column damage indices.

Chapter 7: Damage Assessment of a 6-storey RC Frame – Phase II of EIGM

Table 7.2 -Column damage indices for both phases

Storey	Phase I	Phase II
Roof	0.000	0.030
ST6	0.000	0.137
ST5	0.000	0.006
ST4	0.000	0.005
ST3	0.000	0.005
ST2	0.007	0.035

Figure 7.7 – Time histories of column damage index at the different storeys

Based on previous experimental research for non-seismically designed buildings in Singapore, joint failure at the 6th storey level would have occurred due to a 3.2 % inter-storey drift ratio (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003). Based on damage assessment method by Park and Ang (1985), the maximum damage index of 0.137 for the column below the 6th storey is only

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Chapter 7: Damage Assessment of a 6-storey RC Frame – Phase II of EIGM

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considered as moderate. Thus, the moderate damage level obtained based on Park and Ang (1985) methodology does not capture the damage arising from the joint failure based on experimental research (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003). It can be seen that the level of damage determined by Park and Ang (1985) method does not reflect joint failure. In order to reflect the damage arising from joint failure, the inter-storey drift ratio based on experimental observations (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003) would then have to be adopted.

Since the bending moment and the shear force are the highest at the base columns, a closer investigation of the Phase II bending moments and shear force is undertaken. While Figure 7.8 shows the time histories of the absolute value of shear stress of the three columns, Figure 7.9 shows that of the absolute value of normalized moment of the three columns. Normalized moment is taken as the absolute moment expressed as a fraction of the ultimate moment. The three columns are identified as the exterior (EXT) columns and the interior (INT) with EXT-left and EXT-right. From Figures 7.8 and 7.9, the observations made are as follows:

- The highest shear stress in Phase II can be seen to be experienced by the INT column, which exceeds the ACI shear strength but not the Priestley's shear strength.
- The shear stresses of the EXT columns have not exceeded the ACI shear strength nor the Priestley's shear strength.

All three columns experience bending moments that are greater than the cracking levels and slightly above the yielding levels.

Figure 7.8 – Time histories of shear stress at ground storey columns

Chapter 7: Damage Assessment of a 6-storey RC Frame – Phase II of EIGM

Figure 7.9 – Time histories of normalized bending moment at ground storey columns

7.4 DISCUSSIONS

In this chapter, the response of a frame to EIGMs during Phase II has been determined by modeling the global modes only. The response analysis of the frame structure modeled with global modes has been carried out for both Phase I and Phase II. The following observations can be made from this study:

- The structural displacement of the 6-storey RC frame is larger during Phase II than in Phase I.
- Also, Park and Ang's damage assessment model does not consider the possibility of joint failure. Thus, for the next section, the 3% inter-storey drift ratio shall be used as indication of joint failure (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003).

8 DAMAGE ASSESSMENT OF A 6-STOREY RC FRAME – PHASE I AND PHASE II OF EIGM

This section aims to present the various types of possible failure modes when a structure is subjected to an EIGM. Here, the failure modes which may occur in Phase I or Phase II will be accounted for. By doing so, one can also establish the lower bound of the EIGMs that will cause failure to structure. The structure used in the study is the 6-storey RC frame (Figure 7.1) and the ground shock used is the simulated EIGM at 50 m from detonation (Figure 6.9).

8.1 PROCEDURE AND CONSIDERATIONS

The failure modes which may occur in Phase I or Phase II are as follows:

- Phase I: diagonal shear failure and flexural failure
- Phase II: joint failure, diagonal shear failure and flexural failure

For the joint failure criterion, an inter-storey drift ratio exceeding 3% of the storey height will be considered. This is based on experimental observations of similar constructions (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003).

The following procedures are used to determine the response for the damage assessment for each phase:

- Phase I: Modeling the RC frame for response analysis with local and global modes;
- Phase II: The response analysis for Phase II is determined by modeling the frame with global modes only to determine the response at the end of Phase I. This serves as the initial conditions for the free vibration response during Phase II.

The original ground shock used is a simulated EIGM (Figure 6.9) with the following characteristics: (a) a principal frequency of 185 Hz, (b) a representative PPV of 0.91 m/s and (c) a representative PGA of 113 g. By scaling this EIGM, a family of EIGMs with different characteristics can be generated.

8.2 DAMAGE ASSESSMENT FOR PHASE I

From the Phase I response analysis, the following are the EIGMs that led to shear failure in Phase I. Here, shear failure results from exceedance of strength capacity, where Priestley's concrete shear strength capacity of 1.6 MPa was adopted. The cases of Phase I shear failure represent those where exceedance of shear capacity occurred prior to the occurrence of member cracking moment.

Table 8.1 – Shear failure during Phase I response

Principal Frequency (Hz)	Representative PGA (m/s^2)	Representative PPV (m/s)
360	555	0.23
270	333	0.18
180	555	0.45
72	555	0.90
90	500	1.02

The boundary of shear failure in Phase I is developed from extrapolation of the data featured in Table 8.1 and shown in various representations in Figure 8.1. Here, the boundary of Phase I shear failure is a regression line expressing representative PPV in terms of principal frequency (Figures 8.1a). This boundary defines the minimum representative PPV that will cause shear failure in Phase I for a given principal frequency. The representative PPV is related to the input impulse of an EIGM. Similar results are shown in PGA (Figure 8.1b). Based on Figure 8.1, the following observations can be made:

- With a lower principal frequency (less than 100 Hz), EIGMs that may cause Phase I shear failure require a rapidly increasing PPV with decreasing principal frequencies. This means that increasingly larger input impulse is required to cause shear failure for EIGMs with lower principal frequencies. The local mode response is less significant when the EIGM has a lower principal frequency. Thus, increasingly larger input impulse is required.

Chapter 8: Damage Assessment of a 6-Storey RC Frame- Phase I and Phase II of EIGM

- With a higher principal frequency (more than 100 Hz), EIGMs that may cause Phase I shear failure has a decreasing PPV with increasing principal frequency. The local mode response to EIGMs is more significant with a higher principal frequency. This results in an increased shear force for EIGMs of higher principal frequency. Thus, the minimum PPV required to cause Phase I shear failure under an EIGM is lower. Also, this suggests that the minimum input impulse required to cause Phase I shear failure is lower under an EIGM with a higher principal frequency.
- From Figure 8.1b, it can be seen that the minimum PGA required to cause Phase I shear failure is about 500 m/s². In other words, 500 m/s² is the minimum acceleration, which is required to produce the inertial force that would lead to Phase I shear failure.

Figure 8.1 - Phase I shear failure data and extrapolation

8.3 DAMAGE ASSESSMENT FOR PHASE II

This section refers to the results from the nonlinear response analysis, without the local modes. The results as discussed are only applicable when Phase I shear failure does not occur, as outlined in the earlier section. Here, the focus will be on the global damage, diagonal shear failure and flexural failure during Phase II. Furthermore, joint failure will also be highlighted for Phases I and II.

8.3.1 GLOBAL DAMAGE ASSESSMENT

For the Phase II response analysis, the global damage indices are shown in Figure 8.2. Based on the results shown in Figure 8.2, the following observations can be made:

- The boundaries of regions with different levels of damage are shown (Figure 8.2). The damage index of 0.4 is taken as the boundary separating regions of Moderate damage and Severe damage. The damage index of 1.0 is taken as the boundary for Severe damage zones and Collapse.
- For a given principal frequency, increasing representative PPV of the EIGM leads to a higher level of damage determined (Figure 8.2a).
- For a give principal frequency, increasing representative PGA of the EIGM leads to a higher level of damage determined (Figure 8.2b)

However, the damage assessment based on by Park and Ang (1985) method does not consider the effect of joint failure, which shall be considered later.

Figure 8.2 - Phase II global damage indices and damage levels

8.3.2 COLUMN SHEAR STRESS

The interior column at the first storey experiences the largest shear stress, as was earlier discussed, and so the discussion shall now be made on the effect of changing acceleration and frequency multipliers on this column's shear stress. The interior column shear stress for Phase II is shown in Figure 8.3. Using Priestley's shear strength of 1.6 MPa as a shear failure criterion, regions of "shear failure" and "no shear failure" can be mapped onto the results of various scaled EIGMs. The following observations can be made:

Chapter 8: Damage Assessment of a 6-Storey RC Frame- Phase I and Phase II of EIGM

- For a given principal frequency of EIGM, increasing ground input energy (i.e. higher PPV) led to increasing amount of shear stress induced in the column member during Phase II (Figure 8.3a). Base on the almost horizontal Phase II shear failure line, it can be seen that Phase II shear failure is almost independent of the principal frequency, for a given principle PPV.
- For a given principal frequency of EIGM, increasing inertial force (i.e. higher PGA) led to increasing amount of shear stress induced in the column member during Phase II (Figure 8.3b).

Figure 8.3 - Phase II Shear Failure

8.3.3 COLUMN BENDING MOMENT

For the Phase II response, the bending moment is determined for the interior column at the 1st storey, subjected to the scaled EIGMs. The bending moment induced in the column is then used to provide two zones of column bending moment response to the scaled EIGMs. These zones are “yielding” and “failure” as

Chapter 8: Damage Assessment of a 6-Storey RC Frame- Phase I and Phase II of EIGM

shown in Figure 8.4; while “yielding” is based on exceedance of yielding moment, “failure” is based on exceedance of ultimate moment. The boundary lines are drawn in terms of EIGMs leading to exceedance of yielding moment and ultimate moment. Based on the results shown in Figure 8.4, the following observations can be made:

- These two boundary lines are almost horizontal in Figure 8.4a. This suggests that the column bending moment is nearly independent of the principal frequency. The dependence of the column bending moment on the principal frequency is because the response during Phase II is associated with the global modes. Changes in the principal frequency of the EIGMs have a greater effect on the response in Phase I than in Phase II.
- For a given principal frequency of EIGM, increasing inertial forces (i.e. representative PGA) leads to increased bending moment and so failure by flexural bending.

Figure 8.4 - Phase II column yielding and failure

8.3.4 INTER-STOREY DRIFT RATIO AND JOINT FAILURE

Here, an analysis for joint failure (JF) shall be presented at the 2nd storey level (ST2) and the 6th storey level (ST6). Joint failure criterion is defined as a 3% inter-storey drift ratio based on experimental observations of similar constructions (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003). The inter-storey drift ratios for ST2 and ST6 were determined. The analysis of the nonlinear results for joint failure considered both Phase I and Phase II. It is believed that the inter-storey drift results more from the global mode response than the local mode response. Thus, although the nonlinear response analysis did not include the local mode response, it is assumed that the local mode response does not alter significantly the inter-storey drift response in Phase I. The zones for joint failure for ST2 and ST6 are shown in Figure 8.5. The observations on joint failure are as follows:

- For a given representative PPV (e.g. PPV of 2 m/s), an EIGM of lower principal frequency may lead to joint failure in Phase I at ST2 (Figure 8.5a). On the other hand, an EIGM of a higher principal frequency may lead to joint failure in Phase II at ST2.
- For EIGMs with a principal frequency higher than 100 Hz, a higher representative PPV may lead to joint failure in Phase II at ST2, while a lower representative PPV may lead to joint failure in Phase II at ST6. Thus, for a given principal frequency higher than about 100 Hz, joint failure in Phase II at ST6 precedes that at ST2 as PPV increases. This can be attributed to the first mode shape which shows a side-sway mechanism at ST6. Thus, the more critical Phase II joint failure shall be considered at ST6 and not ST2.
- For a given inertial load (i.e. PGA) of the EIGM, increasing principal frequencies leads to joint failure in Phase II (Figure 8.5b).

Figure 8.5 - Joint failure for ST2 and ST6 in Phase I and Phase II

8.3.5 COMBINED FAILURE ZONES FOR PHASE II

For the 6-storey RC frame, the earlier sections have allowed for the separate mapping of the zones of different failure mechanisms for Phase II, due to the scaled EIGMS. The failure mechanisms are:

1. Phase II diagonal shear failure (SF) based on Priestley strength capacity,
2. Phase II joint failure (JF) based on the experimental observations of similar constructions (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003). In Figure 8.5, it was shown that joint failure during Phase II is more critical at ST6 than ST2. For the purpose of the combining failure

Chapter 8: Damage Assessment of a 6-Storey RC Frame- Phase I and Phase II of EIGM

mechanisms during Phase II due to the scaled EIGMs, only joint failure at ST6 shall be considered.

3. Phase II flexural failure (FF) based on exceedance of the bending moment capacity.

These three failure mechanisms are consolidated onto a single map as shown in Figure 8.6. This is to allow the choosing of governing failure mechanisms for Phase II due to the scaled EIGMs. The failure mechanisms are presented in terms of principal frequencies and representative PPVs of the scaled EIGMs.

Figure 8.16a shows the direct combination of the following failure mechanism during Phase II due to the scaled EIGMs:

- Phase II SF from Figure 8.3,
- Phase II FF from Figure 8.4 and
- Phase II JF at ST6 from Figure 8.5.

Figure 8.16a shows the lower bound of the scaled EIGMs that cause the various failure mechanisms. As can be seen, the Phase II FF is embedded within the Phase II SF. Thus, it can be said that Phase II SF is more critical than Phase II FF. or the scaled EIGMs, when the principal frequency is less than about 100 Hz and the representative PPV is larger than 1 m/s, increasing PPV causes the change from Phase II SF to Phase II JF. For the scaled EIGMs, when the principal frequency is greater than 100 Hz, increasing PPV causes the change from Phase II JF to Phase II SF. With this, a refined combination of the failure mechanisms for Phase II due to scaled EIGMs by choosing Phase II SF as more critical than Phase II FF, Figure 8.16b is produced.

Figure 8.6 - Damage characteristics from Phase II response analysis

8.4 COMBINING PHASE I AND PHASE II RESULTS

In this section, the failure mechanisms for Phase I is combined with Phase II to produce the damage characteristics of a 6-storey RC frame due to EIGMs (Figure 8.7). The failure mechanisms are presented in terms of the principal frequencies and representative PPVs of the scaled EIGMs. First, the failure mechanisms in Phase I alone is combined which includes Phase I SF (Figure 8.1) and Phase I JF at ST2 (Figure 8.5) and shown in Figure 8.7a.

Chapter 8: Damage Assessment of a 6-Storey RC Frame- Phase I and Phase II of EIGM

Following this, the combined failure mechanisms in Phase I (Figure 8.7a) is integrated with the combined failure mechanisms in Phase II (Figure 8.6c) and presented in Figure 8.7b. From this, EIGMs leading to shear failure in Phase I and also joint failure in Phase II can be seen in the upper right-hand corner (e.g. EIGM with principal frequency of 200 Hz and representative PPV of 2 m/s). Through the combination of the results, such a phenomenon may not be possible as shear failure in Phase I would have occurred first. The results shown in Figure 8.7b does not take into consideration the chronological effect of failure occurring during Phase I or Phase II. By this, a structure that experiences failure in Phase I would have failed, and any failure as determined in Phase II may not be experienced. A failure map should therefore not consider failure mechanisms in Phase II when there is failure in Phase I.

For EIGMs that lead to failure mechanisms in both Phase I and Phase II, Phase II failure is neglected for a more refined combination of the failure mechanisms (Figure 8.7c). The results as shown in Figure 8.7c lead to a combined failure map representing the damage characteristics of the 6-storey RC frame due to EIGMs in Phase I and Phase II. Based on the results shown in Figure 8.7c, the following observations can be made:

- There is a lower bound for the failure mechanisms in both Phase I and Phase II. For principal frequencies ranging from 10 Hz to about 70 Hz, this lower bound is constant at about 1.5 m/s. For principal frequencies ranging from about 70 Hz to about 700 Hz, this lower bound has PPVs ranging from about 1.5 m/s to 0.1 m/s, respectively. Scaled EIGMs with representative PPV and principal frequencies above this lower bound may induce failure in the 6-storey RC frame. However, scaled EIGMs with representative PPV and principal frequencies below this lower bound do not induce in the 6-storey RC frame, for any of the failure mechanisms considered.
- Failure mechanisms in Phase II, which are Phase II SF and Phase II JF, only occur due to scaled EIGMs with principal frequencies less than about 70 Hz. Furthermore, the PPV of the scaled EIGMs associated with these failure mechanisms is less than about 3 m/s. Thus, EIGMs with lower principal

Chapter 8: Damage Assessment of a 6-Storey RC Frame- Phase I and Phase II of EIGM

frequencies and PPVs lead to failure during Phase II. On the other hand, EIGMs with higher principal frequencies and PPVs lead to failure during Phase I.

In addition, a tripartite plot of failure mechanisms in terms of principal frequency, PPV and PGA is shown for the 6-storey RC frame in Figure 8.8. This is presented similar to the tripartite plot of response spectra in earthquake engineering. While the PPV is proportional to the input impulse, the PGA is proportional to inertial forces. Lastly, the principal frequency is proportional of the rate of loading. With this, the parameters of PPV, PGA and principal frequency can be linked more intuitively to input impulse, inertial load and rate of loading, respectively. Base on the results shown in Figure 8.8, the following observations about the 6-storey frame subjected to EIGM loads, can be made:

- An EIGM at the lower left-hand corner is one with a low rate of input impulse and low rate of inertial load. On the other hand, an EIGM at the upper right-hand corner is one with a high rate of input impulse and high rate of inertial loading.
- In general, it can be seen that EIGMs with a larger PPV leads to a failure mode being experienced during Phase I. An EIGM with a PPV of 1.5 m/s and principal frequency of 50 Hz leads to diagonal shear failure during Phase II. Another EIGM with a higher PPV of 3 m/s but with the same principal frequency leads to diagonal shear failure in Phase I.
- Also, for EIGMs with similar PPVs, those with a higher principal frequency lead to failure in Phase I while those with a lower principal frequency lead to a failure in Phase II. This is due to the increased contribution from the local modes in Phase I when the principal frequency of the EIGM is higher.
- Thus, the three parameters that can be used to describe the damaging potential of EIGM are its principal frequency, representative PPV and representative PGA, which are conveniently shown in a tripartite plot.

Chapter 8: Damage Assessment of a 6-Storey RC Frame- Phase I and Phase II of EIGM

Figure 8.7 - Damage characteristics of 6-storey RC frame due to scaled EIGMs

Figure 8.8 - Tripartite plot of damage characteristics of a 6-storey frame due to EIGMs

There is a unique feature as presented by the damage assessment graph for the 6-storey RC frame. It is capable of displaying the various possible modes of failure that would be experienced by the 6-storey RC frame when subjected to EIGMs.

With the possible failure mode recognized, measures can be taken when necessary to prevent the possible failure. These would include enhancing the corresponding strength to avoid the possible failure. For instance, in the design of an RC frame subjected to the family of EIGMs, a damage assessment technique such as the above can be carried out. If it reflects Phase II joint failure, then the designer should enhance the joint strength through proper joint detailing.

8.5 COMPARISON: 2-STOREY AND 6-STOREY RC FRAMES

Here, the scaled EIGMs that lead to the failure of the 6-storey RC frame have been identified (Figure 8.7c and 8.8). Similarly, the scaled EIGMs that lead to failure of a 2-storey RC frame has also been identified (Pan et al, 2004b). Both results have been presented in terms of the representative PPV and principal frequency, and are shown in Figure 8.9. From Figure 8.9, it can be seen that while the lower bounds of the failure regions are similar, the scaled EIGMs that can cause structural failure of the 2-storey RC frame have a higher PPV.

Figure 8.9 – Comparison of lower bound of failure in the 2-storey and 6-storey RC frames

The taller RC frame of 6-storey has a lower structural frequency than the shorter 2-storey RC frame. When subjected to the same EIGM, the 6-storey RC frame would display more significant global mode than the 2-storey RC frame. The lighter and

Chapter 8: Damage Assessment of a 6-Storey RC Frame- Phase I and Phase II of EIGM

stiffer 2-storey RC frame may therefore resist EIGMs of a higher PPV at the same principal frequency.

By comparing the trend of structural failure between the 2-storey and 6-storey RC frames, a relationship in the resistance of RC frames to EIGMs and the height of the RC frame can thus be postulated. It can be said that a lighter and stiffer RC frame may resist EIGMs of a higher PPV at the same principle frequency, provided that the RC frames are based on the same design code. Figure 8.10 indicates the postulate trend.

Figure 8.10 - Postulated relationship between failure of RC frame and height of RC frame

8.6 DISCUSSIONS

This chapter set out to highlight the possible failure modes related to ductility and toughness issues of an RC frame when subjected to an EIGM. A plot of the damage characteristics for a 6-storey RC frame was made by combining the various failure mechanisms when subjected to scaled EIGMs. From there, it is noted that:

- Increasing PPV and principal frequency values of an EIGM can lead to a failure mode during Phase I. This type of failure is more brittle and related to insufficient toughness in the structure.

Chapter 8: Damage Assessment of a 6-Storey RC Frame- Phase I and Phase II of EIGM

- However, reducing PPV leads to a failure mode during Phase II. This type of failure is more related to the lack of ductility.

Recalling how masses can filter high-frequency waves and in view of the above observations, the following implications can be made:

- When a building is sited closer to a detonation, the EIGM that reaches the building would be of a higher frequency. As a result, the building may experience failure during Phase I. This means that the building would have to be tough enough in order to prevent the brittle type of failure.
- On the other hand, for a building that is sited farther from the detonation, the frequency of the waves reaching the building would be lower due to the filtering and attenuation of waves by the soil/rock medium. Therefore, buildings sited farther from the detonation would have to be more ductile in order to prevent the failure associated with global deformations.

For the damage assessment of buildings around ammunition storages, authorities can do a survey of the buildings in the vicinity. Then, by categorizing the buildings, they can estimate the damage characteristics for each building category similar to that shown in Figures 8.7 and 8.8. This would allow an understanding of the possible failure mechanisms in order to take preventive measures when necessary.

From the parametric studies for the effect of scaling the peak ground acceleration and the dominant frequency of an EIGM at 50 m, the conclusions are:

1. A higher ground input energy (i.e. higher representative PPV) leads to a higher response and thus damage levels;
2. The displacement response during Phase II is generally higher than that during Phase I;
3. Response-based damage assessment techniques does not adequately reflect the damage associated with joint failure;
4. For a given EIGM, joint failure would occur in a RC frame before flexural failure of the members can take place;

Chapter 8: Damage Assessment of a 6-Storey RC Frame- Phase I and Phase II of EIGM

5. While increasing shear strength can toughen an RC frame, joint failure may then become the failure mechanism of the frame;
6. Figure 8.8 summarized the damage mechanisms of the 6-storey RC frame in terms of the principle PPV and the principle frequency;
7. The four main damage mechanisms (Phase I SF, Phase I JF, Phase II SF and Phase II JF) can be clearly located on Figure 8.8; and
8. The results seem to suggest that lighter and shorter RC frames may resist EIGMs with a higher PPV.

9 CONCLUSIONS AND RECOMMENDATIONS

9.1 CONCLUSIONS

In this thesis, the research area is on RC structures that are subjected to explosions-induced ground motions (EIGMs). The RC frames studied are non-seismically designed. The research work focuses on two areas which are related to response analysis and damage assessment.

Within the response analysis of structures subjected to EIGMs, the beam theories of Euler-Bernoulli and Timshenko are compared. This is carried out with a fixed-fixed column and a fixed-free column subjected to triangular EIGM loads. In addition, the effect of boundary condition on induced forces is studied. Having compared the two beam theories, an approximate method for the response analysis during the major shock duration (Phase I) is proposed. This proposed method is extended to MDOF systems subjected to general triangular EIGM loads. This proposed approximate method has been shown to produce accurate response results but requires less computational effort. Also, as the approximate method is able to separate the response analyses during the major shock duration (Phase I) and the ensuing duration (Phase II), computational effort can be saved for the response analysis of RC frames subjected to EIGMs.

Following this, a method for damage assessment during Phase I response is proposed. This proposed method for damage assessment considers strength enhancement due to strain rate effects. Also, the damage assessment method determines the type of shear failure that the structure may experience, which are diagonal shear failure and direct shear failure. The damage assessment method is demonstrated for a fixed-fixed column subjected to a simulated EIGM load.

Having demonstrated the proposed method for damage assessment during Phase I, a simplified methodology is proposed for the damage assessment of a multi-storey RC frame. This proposed simplified method involves performing the damage assessment of a multi-storey RC frame by performing shear failure analysis on the

Chapter 9: Conclusions and Recommendations

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base column. For a 2-storey RC frame subjected to scaled EIGMs, the simplified method is applied by performing the proposed damage assessment on a fixed-fixed column from the RC frame. Both the RC frame and the scaled EIGMs are similar to those used in an earlier work (Pan et al, 2004b). The results from the damage assessment of the fixed-fixed column are similar to those from the material-level modeling (Pan et al, 2004b). Thus, the simplified methodology for damage assessment by performing the damage assessment of a column at the base of a 2-storey RC frame is feasible. This simplified methodology reduces computational effort.

Also, possible failure mechanisms in Phase I and Phase II are mapped for a 6-storey RC frame subjected to scaled EIGMs. The failure mechanisms considered were Phase I shear failure, Phase I joint failure, Phase II flexural failure, Phase II shear failure and Phase II joint failure. Here, joint failure was defined based on experimental research of similar constructions (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003). The map of the failure mechanisms allows for the identification of possible failure mechanisms in both Phase I and Phase II when an RC frame is subjected to EIGMs. By doing so, measures can be taken when necessary to prevent the possible failure.

By comparing the damage assessment of a 2-storey RC frame to a 6-storey RC frame, it is postulated that lighter and shorter RC frames can resist EIGMs of a higher PPV, provided the RC frames are based on the same design code.

Throughout the study, numerical studies of structures subjected to EIGM have been conducted. The observations are as follows:

- When a structure is subjected to an EIGM, high-frequency local mode response is evident during Phase I
- During Phase I, large shear forces are induced accompanied by significantly small displacements. As a result of that, the structure is put through a *toughness test*.

Chapter 9: Conclusions and Recommendations

- During Phase II, the structure experiences larger deformations. While the structure undergoes larger deformations during Phase II, the structure is put through a *ductility test*.

These observations agree with those by other researchers (Lu et al, 2001; Pan et al, 2001; Ma et al, 2002; Pan et al, 2002; Dhakal and Pan, 2003; Pan et al 2003a; Pan et al, 2004b).

The following sections describe the findings related to response analysis and damage assessment in greater details.

9.1.1 RESPONSE ANALYSIS

Related to the response analysis of structures to EIGMs, the Euler Bernoulli and Timoshenko beam theories were compared and a proposed method for response analysis during Phase I is made.

Beam theories

A comparison between the Euler-Bernoulli beam theory and the Timoshenko beam theory is made (Chapter 5). For the comparison, a fixed-fixed column and a fixed-free column were subjected to a triangular EIGM load. In each case, the column is modeled using finite element modeling of Euler-Bernoulli beam theory and Timoshenko beam theory.

It is shown that Euler-Bernoulli elements can produce similar response results as compared to Timoshenko elements. However, this is only true when *using a sufficient number of Euler-Bernoulli elements and distributing the mass of the member* at intermediate points along the member. By doing so, the wave will propagate at finite speeds along the member's length.

From the analyses of the fixed-fixed and fixed-free columns subjected an EIGM load, it is observed that the displacement is the largest at the mid-points of the columns due to local mode response. However, *the shear force and bending*

Chapter 9: Conclusions and Recommendations

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moments induced are the largest at the base for both columns. Thus, the failure would most likely be at the column base.

In the analyses of the fixed-fixed and fixed-free columns, it has been shown that the boundary conditions at the top of the column have negligible effects on the forces determined at the column base. This implies that the damage assessment for a multi-storey RC frames can be performed by conducting a damage assessment on the column at the first-storey level.

Proposed approximate method

The proposed approximate method has been developed for SDOF systems and extended to MDOF systems (Section 6.1). It has been developed for general triangular EIGM loads. Before developing the proposed method, it has been shown that the effect of structural damping is negligible for an SDOF system subjected to short duration EIGM loads (Figure 6.2).

The impulse-momentum relationship (Clough and Penzien, 1993) is adopted for the approximate displacement and velocity responses. However, the displacement and velocity responses as determined will be overestimated as this neglect some terms in the dynamic equilibrium equation. Thus, correction factors are developed to improve the approximate velocity and displacement responses (Section 6.1.1). The correction factors are based on the closed-form solution for a SDOF system subjected to a general triangular EIGM load. The approximate responses with the use of the correction factors are determined for a SDOF system subjected to a general triangular load. These results are shown to be similar to those determined by the closed-form solutions.

A feasibility study is performed related to the use of approximate responses to serve as the initial conditions for the response analysis of Phase II. It is shown that by doing so, the response results in Phase II are similar to those obtained by the closed-form solutions (Figure 6.8).

Chapter 9: Conclusions and Recommendations

Later, the proposed approximate method for response analysis of Phase I, is extended to an MDOF system. The response of a 2-storey RC frame when subjected to an idealized form of a simulated EIGM load, is analyzed using the proposed approximate method. The response results as determined are significantly accurate (Figure 6.10).

From the numerical studies, the proposed approximate method has been shown to have the following features:

- It can be used to include local mode contribution which is significant for the structural response during Phase I.
- It can produce a time history for the response during Phase I, which is important since the peak values for induced forces takes place during the loading duration.
- It produces accurate results for structural response during Phase I
- It can be used to separate the response analysis of Phase I from that of Phase II. Thus, the proposed approximate method for response analysis can be used to determine the response during Phase I and also to separate the response analyses during Phase I and Phase II.

Hence, the use of the proposed approximate method can lead to reduced computational effort.

9.1.2 DAMAGE ASSESSMENT

Related to the damage assessment of structures to EIGMs, the damage assessment of columns are undertaken using a proposed damage index for Phase I. This is further analyzed via a proposed simplified methodology for damage assessment. In addition, the failure mechanisms for Phase I and Phase II are mapped for a 6-storey RC frame subjected to scaled EIGMs. The mapping of the failure mechanisms for the 6-storey RC frame establishes the lower bound of failure mechanisms and also identifies the possible failure mechanisms when the RC frame is subjected to EIGMs.

Proposed Damage Assessment during Phase I

A method for damage assessment during Phase I is proposed (Section 6.2). The proposed method accounts for strength-enhancement effects due to strain rate effects. This is crucial as it has been shown that the strength can increase by as high as 40% for a column subjected to EIGM loads. For this, the proposed damage index shall be taken as the maximum of two ratios, each of which is the ratio of the induced force or bending moment to the respective strength capacity. The proposed method for damage assessment determines the possible modes of shear failure, which are diagonal shear failure and direct shear failure. The determination of the possible modes of shear failure is based on the research by Li and Jones (2000). A procedure for this proposed method is shown in Figure 6.12.

The proposed method for damage assessment is used to study a fixed-fixed column subjected to a triangular EIGM load (Section 6.2.2). By subjecting the fixed-fixed column to scaled versions of the triangular EIGM load, the modes of failure by diagonal shear (Section 6.2.3) and direct shear (Section 6.2.4) are determined by the proposed method for damage assessment. It is observed that for EIGMs of a given PPV, an EIGM that would cause direct shear failure would be of a higher principal frequency compared with one which may cause diagonal shear failure.

Simplified Methodology for Damage Assessment of Multi-storey RC frames

In a multi-storey RC frame, the shear force and bending moment induced are the largest at the column base located at the first-storey level. This column thus becomes the critical member for determining structural failure. In Chapter 5, it has been observed that the boundary condition at the top of a column has negligible effect on the forces induced at the column base.

With this, it is proposed that the damage assessment of a multi-storey RC frame be conducted by assessing the damage on the column at the first storey level of the RC frame (Section 6.3). Here, the simplification can reduce computational effort substantially.

Chapter 9: Conclusions and Recommendations

This simplified methodology for damage assessment of a multi-storey RC frame is performed on a 2-storey RC frame. Scaled versions of a simulated EIGM load are used. In the damage assessment of the column, structural-level modeling is used. The damage assessment results by the proposed simplified methodology have been compared with those from the material-level modeling of the same RC frame (Pan et al, 2004b). The results from these two methods have been shown to be similar (Figure 6.24). Further, a distinction is made between diagonal shear failure and direct shear failure when using the proposed simplified methodology. This distinction was not made in the results obtained by Pan et al (2004b). This suggests that a simplified damage assessment for a single column can be performed to determine the possible failure mechanisms within a multi-storey RC frame.

Mapping of failure mechanisms in Phase I and Phase II

The possible failure mechanisms in Phase I and Phase II are mapped for a 6-storey RC frame subjected to scaled EIGMs (Chapter 8). For a non-seismically designed RC frame in Singapore, experimental studies have shown that at these levels of inter-storey drifts, the RC frame will experience failure at the beam-column joint (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003). The possible failure mechanisms considered are Phase I shear failure, Phase I joint failure, Phase II flexural failure, Phase II shear failure and Phase II joint failure. By identifying the possible failure mechanisms, measures can be taken where necessary to prevent the failure.

In Phase I, the RC frame is modeled for local and global modes. During Phase II, the RC frame is modeled for global modes only. This approach is based on the negligible response of local modes of a structure during Phase II.

Failure mechanisms in Phase I

For the 6-storey RC frame subjected to scaled EIGMs, it has been shown that increasing PPV of the EIGM at the same principal frequency may lead to shear failure during Phase I (Figure 8.1). EIGMs of low principal frequencies and low PPV may lead to joint failure during Phase I (Figure 8.5).

Chapter 9: Conclusions and Recommendations

Failure mechanisms in Phase II

A response-based method is used for the damage assessment of a 6-storey frame during Phase II (Section 8.3). The damage assessment used is that of Park and Ang (1995). It is seen that the structure experiences large inter-storey drifts during Phase II. However, the damage level as determined using Park and Ang (1985) was only considered to be *moderate* (Figure 7.6 and 7.7). It can thus be seen that this damage assessment method does not reflect properly the joint failure phenomenon.

From the damage results during Phase II, it has been shown that failure by flexural bending will be accompanied by failure by diagonal shear failure; failure by diagonal shear may however be experienced without failure by flexural bending (Figure 8.6b). This suggests that failure by diagonal shear is more critical than that by flexural bending for non-seismically designed RC frames. Nevertheless, in preventing the failure by diagonal shear via enhancing the shear strength capacity, the moment strength capacity would inherently increase too.

Combined failure mechanisms in both Phase I and Phase II

The failure mechanisms in Phase I and Phase II of the 6-storey RC frame, due to the scaled EIGMs, are combined (Figures 8.7 and 8.8). It has been shown that EIGMs of lower PPV and lower principal frequencies may lead to failure during Phase II. On the other hand, EIGMs of higher PPV and higher principal frequencies may lead to failure during Phase I.

Comparison of failure mechanisms in a 2-storey and a 6-storey RC frame

The lower bound of failure mechanisms of a 2-storey and a 6-storey RC frame are compared (Figure 8.9). From this, it is postulated that lighter and stiffer RC frames can resist EIGMs of a higher PPV, provided the RC frames are based on the same design code.

9.1.3 FRAMEWORK FOR DAMAGE ASSESSMENT

Based on the work in this thesis, a framework for the damage assessment of RC frames subjected to EIGMs is outlined. The framework is illustrated in Figure 9.1 and described as follows:

- Check for the possibility of shear failure during Phase I. For a multi-storey frame, a quick damage assessment can be performed for the structure by investigating the columns at the first-storey level. This simplified method of analysis has been shown to provide results similar to those from material-level modeling (Pan et al, 2004b). The column can be simplified with a fixed-fixed boundary condition. The shear force and bending moment induced at base are the largest within the multi-storey frame. It has been shown that boundary conditions do not significantly affect the induced forces determined at this location. While Timoshenko beam theory has been suggested for modeling a structure under impulsive loads, it has been demonstrated that using a sufficient numbers of element with Euler-Bernoulli beam theory produces similar response results.
- For Phase I, the response analysis can be determined using the proposed approximate method developed for an MDOF system. For Phase I, the proposed damage assessment method, which considers the strength enhancement due to strain rate effects, can be used to determine the possible failure modes. The failure modes include diagonal shear failure, direct shear failure and flexural failure for this phase. This approach would save computational effort significantly. Thus, a quick check can be made on a multi-storey frame by investigating the toughness of the critical member which is the column at the first-storey level. If the column experiences shear failure during Phase I, increasing the shear strength capacity would be required.
- If the RC frame does not experience shear failure during Phase I, the response at the end of Phase I as determined by the approximate method can be used as the initial conditions to determine the response during Phase II. For this, local modes may not be included as the response is contributed largely by global modes. Damage assessment techniques for Phase II should

Chapter 9: Conclusions and Recommendations

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be response-based and include the beam-column joint failure mode. In this study, it has been shown that the possibility of joint failure for non-seismically designed structures, when subjected to EIGMs, is possible. This has not been considered in earlier analyses.

Figure 9.1 - Illustration of framework for damage assessment

9.2 RECOMMENDATIONS

While a joint model has been proposed by Lowes et al (2004a, 2004b), it however does not always provide analytical results similar to experimental observations. Thus, a joint model that can do this should be developed. Here, the criterion for joint failure is based on experimental observations of similar constructions (Wu et al, 2001; Li et al, 2002; Li et al, 2003; Dhakal and Pan, 2003). By having a developed joint model, joint failure and deformation can be explicitly modeled.

For Phase I, the proposed damage assessment method considers the maximum of the shear damage index and the flexural damage index. However, this may not be accurate. With additional experimental observations, a system of weighting that shear damage index and flexural damage index may be accurate.

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