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SEISMIC PERFORMANCE OF MOMENT-RESISTING CONCRETE FRAMES SUBJECTED TO EARTHQUAKE EXCITATION

FADZLI MOHAMED NAZRI*, PANG SZE YAN

Disaster Research Nexus (DRN), School of Civil Engineering,
Universiti Sains Malaysia, Seri Ampangan, 14300 Nibong Tebal,
Seberang Perai Selatan, Pulau Pinang, Malaysia
*Corresponding Author: cefmn@usm.my

Abstract

In this study, moment-resisting concrete frames (MRCFs) were designed based on Eurocodes 2 and 8, which indicate the seismic provisions and requirements for building design and construction. This study aims to investigate the damage measure of MRCFs subjected to earthquake excitation by pushover analysis (POA) and incremental dynamic analysis (IDA). In POA, inverted triangular lateral load and uniform lateral load patterns were used to produce a base shear–drift curve. In IDA, seven far-field and seven near-field ground motion records were selected to establish the base shear–drift relationship. Structural response and damage measures were examined by the performance-based seismic design limit states. Vision 2000 proposed four performance states, namely, fully operational, operational, life safety (LS), and near collapse. The results showed that the designed structures have low stiffness because all MRCFs failed to meet the LS limit state. The base shear–drift relationship produced a higher demand in IDA than in POA. In POA, the lateral uniform load pattern produced higher demand than the lateral inverted triangular load pattern. In IDA, the far-field effect produced higher demand than the near-field effect. POA approximated IDA accurately at the elastic stage, but the approximation failed after the yield point.

Keywords: Base shear–drift relationship, IDA, MRCFs, PBSD limit states, POA.

1. Introduction

Malaysia is situated in the southern edge of the Eurasian Plate. This country is close to the two most seismically active plate boundaries, namely, the inter-plate boundary between the Indo-Australian and Eurasian Plates in the west and the inter-plate boundary between the Eurasian and Philippine Plates in the east [1]. Large earthquake

Nomenclatures

q	*Behavior factor*
T_1	Natural period (Table 1)

Greek Symbols

λ	Important factor

Abbreviations

EC2	Eurocode 2
EC8	Eurocode 8
IDA	Incremental dynamic analysis
LS	Life safety
MRCF	Moment-resisting concrete Frame
NC	Near collapse
O	Operational
PBSD	Performance-based seismic design
PEER	Pacific earthquake engineering research center
PGA	Peak ground acceleration
POA	Pushover analysis

in and around these boundaries can cause devastating damage to properties and lives. Malaysia is favorably far from the boundary of the Eurasian Plate and is stable and safe from earthquakes. However, buildings on soft soil are occasionally subjected to tremors because of the far-field effects of earthquakes in Sumatra [2]. Earthquakes are common in Malaysia. The effect of seismic action should be considered in building designs in Malaysia to avoid devastating loss of life and property. Therefore, studies on the seismic behavior and relative structural performance of buildings are important.

2. Nonlinear Analysis

Reinforced concrete buildings are common in Malaysia. Residential, dormitory, institutional, office, commercial, industrial buildings, and many other infrastructures are built by using reinforced concrete. The basic skeleton of a framed building comprises beams, columns, and footings. To resist lateral earthquake forces and carry vertical gravity loads, a moment-resisting concrete frame (MRCF) is desirable particularly for medium and high-rise buildings. The concrete frame design used in this study is a simple generic moment-resisting concrete structure. As defined by Alavi and Krawinkler [3], generic frames are based on many simplifying assumptions that do not represent the real condition of structures in all cases.

Three types of MRCF with three, six, and nine stories were individually and economically designed according to Eurocode 2 (EC2) [4] and Eurocode 8 (EC8) [5]. EC2 provides the design code for reinforced concrete frames,whereas EC8 provides the general requirements for earthquake-resistant designs. Each building case has three bays (6 m span) and an identical storey height of 3.3 m. The frame of all structures is assumed to fall into the Type 1 elastic response spectrum for Type A soil and have a peak ground acceleration (PGA) of 0.5 g. The behavior factor (q = 4) for MRCF buildings with medium ductility is used as suggested by EC8. Soil class A is considered to avoid the soil–structure interaction in the analysis. Details on the design assumptions and beam and column specifications are presented in Tables 1-3.

Table 1. Design Parameter for all Cases.

Design Parameter	Value	Unit
Bar diameter	32	mm
Link diameter	10	mm
Cover to reinforcement	25	mm
Concrete strength	30	N/mm²
Steel yield stress	460	N/mm²
T_1 (3storey)	0.47	s
T_1 (6storey)	0.80	s
T_1 (9storey)	1.08	s
Important factor, λ	1	

Table 2. Beam Size and Reinforcements.

No. of Storey	Sizes (mm x mm)	Reinforcement (area/mm²)	Shear Link
3	400 × 700	4T32 (3217)	10 mm links at 150 mm c/c
6	400 × 700	4T32 (3217)	10 mm links at 150 mm c/c
9	450 × 800	4T32 (3217)	10 mm links at 150 mm c/c

Table 3. Column Size and Reinforcements.

No. of Storey	Sizes (mm x mm)	Reinforcement (area/mm²)
3	650 × 650	5T32 (4022)
6	650 × 650	4T32 (3217)
9	650 × 650	3T32 (2413)

2.1. Lateral load patterns

For a realistic performance evaluation, load pattern selection is crucial. Given that no single load pattern can capture the variations in local demands expected in a design earthquake, the use of at least two load patterns that are expected to bound inertia force distributions is recommended [6]. Hence, inverted triangular lateral load and uniform lateral load patterns were used in POA to produce the base shear–drift curve.

2.2. Ground motion records

The Pacific Earthquake Engineering Research Center (PEER) developed a ground motion database that includes the ground motions of shallow crustal earthquakes in active tectonic regimes worldwide. For this study, 14 ground motion records are selected from the database of PEER. All ground motion records have a magnitude of 6.0 to 7.0. Table 4 shows the selected ground motion records for

IDA. J-B distances of <20 and >20 km indicate near-field motion and far-field motion, respectively.

Table 4. Selected Ground Motion Records for IDA.

	J-B Distance (<20 km)		J-B Distance (>20 km)
NGA 0455	Morgan Hill 1984	NGA 0438	Borah Peak, ID-01 1983
NGA 0518	N. Palm Springs 1986	NGA 0470	Morgan Hill 1984
NGA 0550	Chalfant Valley-02 1986	NGA 0528	N. Palm Springs 1986
NGA 0725	Superstition Hills-02 1987	NGA 0555	Chalfant Valley-02 1986
NGA 0810	Loma Prieta 1989	NGA 0724	Superstition Hills-02 1987
NGA 1085	Northridge-01 1994	NGA 0805	Loma Prieta 1989
NGA 1120	Kobe, Japan 1995	NGA 0980	Northridge-01 1994

In IDA, seven far-field and seven near-field ground motion records were selected to establish the base shear–drift relationship.The ground motion records were scaled to a PGA, and analyses were conducted for all records. The PGA was then increased, and the analyses were repeated until the IDA curve was obtained. A full nonlinear time historey analysis was performed for each increment.

2.3. Performance-based seismic design

Structural modeling is conducted by using SAP 2000 software. The results obtained from POA and IDA will be examined to investigate the limit states based on the performance-based seismic design PBSD approach. A critical component of this approach is the definition of performance limit states. Structural deformation, typically inter-storey drift, has been suggested as a relatively easy marker for the level of damage in the structural system. Vision 2000 [7] suggests permissible drifts of 0.2% for fully operational, 0.5% for operational (O), 1.5% for life safety (LS), and 2.5% for near collapse (NC).

3. Results and Discussion

Structural modeling is conducted by using SAP 2000 software. The results obtained from POA and IDA will be examined to investigate the limit states based on the performance-based seismic design PBSD approach. A critical component of this approach is the definition of performance limit states. Structural deformation, typically inter-storey drift, has been suggested as a relatively easy marker for the level of damage in the structural system. Vision 2000 [7] suggests permissible drifts of 0.2% for fully operational, 0.5% for operational (O), 1.5% for life safety (LS), and 2.5% for near collapse (NC).

3.1. Pushover Analysis (POA)

The base shear produced by lateral uniform load is generally greater than the base shear produced by lateral inverted triangular load as shown in Figs. 1-4. For example, the base shear of the six-storey frame under uniform loading is 1324.3 kN

(Fig. 4, which is greater than the 1066.8 kN base shear of the same frame under triangular loading (Fig. 3 and 4). The same trend can be observed in three- and nine-storey frames. This outcome can be explained by the demand emphasis of uniform load patterns in lower stories than in upper stories and the magnification of the relative importance of storey shear forces compared with overturning moments [6]. By contrast, an inverted triangular load pattern emphasizes the demands in upper floors, thus resulting in the magnification of overturning moments.

In all POA curves, three-storey frames under uniform and triangular loadings achieve LS limit states with drift percentages of 1.52% and 1.51%, respectively (Fig. 1 and 2). However, six- (Fig. 3 and 4) and nine-storey frames (Fig. 5 and 6) under both loading patterns fail to meet the LS limit state. The drift percentages of six- and nine-storey frames lie only between the O and LS limit states. Hence, the drift of the structure with a high number of stories decreases if the lateral stiffness has an insignificant increase.

In terms of displacement, nine-storey frames have the largest displacement compared with six- and three-storey frames. Therefore, higher buildings have higher displacement tendencies. In POA, the displacement at the roof top is generally the largest. However, for the nine-storey frame under uniform load (Fig. 6), the condition shown an example of a soft-storey case. A soft storey is defined as a storey in a building with substantially less stiffness than other stories. A soft storey has an inadequate shear resistance or ductility to resist stress [8].

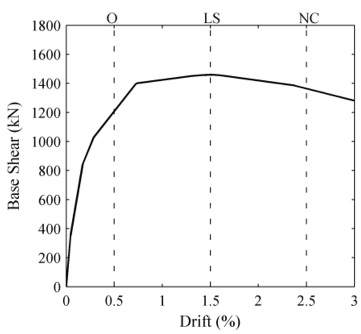

Fig. 1. POA Curve for the 3-Storey Frame under Triangular Loading.

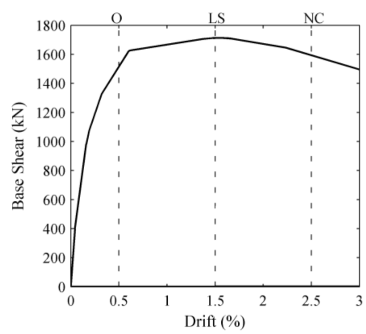

Fig. 2. POA Curve for the 3-storey Frame under Uniform Loading.

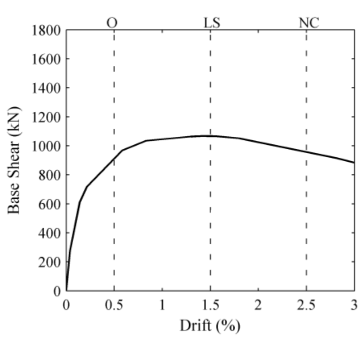

Fig. 3. POA Curve for the 6-storey Frame under Triangular Loading.

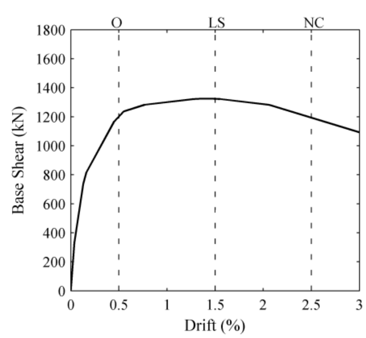

Fig. 4. POA Curve for the 6-storey Frame under Uniform Loading.

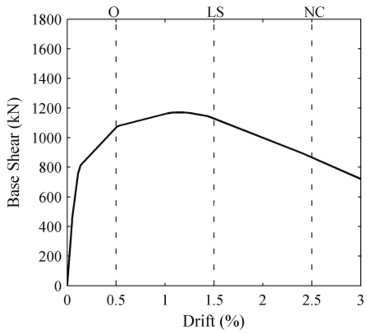

Fig. 5. POA Curve for the 9-storey Frame under Triangular Loading.

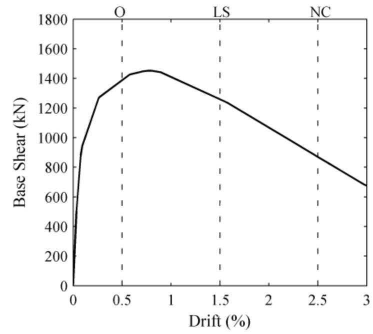

Fig. 6. POA Curve for the 9-storey Frame under Uniform Loading.

3.2. Incremental dynamic analysis (IDA)

The IDA curve for the three-storey frame under the near-field effect (Fig. 7) indicates that the frame shows its elasticity at a base shear of 1418 kN.The frame displaces 0.028 m, which is 0.29% in drift. According to Vision 2000 [7], the drift percentage is within the O limit state. When the PGA reaches 0.5 g, the frame structure collapses at a base shear of 2014 kN.The frame displaces 1.072 m, which is 10.83% in drift. The drift percentage exceeds the LS and NC limit states.

IDA curve starts to soften at approximately 0.4 g by displaying a tangent slope less than the elastic slope.The IDA curve then hardens with a local slope higher than the elastic slope. The hardening phenomenon observed in Fig. 7 can be extreme for some records to return the structure from global collapse. This phenomenon is called structural resurrection [9]. In Fig. 8, the three-storey frame was subjected to a far-field effect, the frame shows its elasticity at a base shear of 1862 kN.The frame displaces 0.032 m, which is 0.32% in drift. According to Vision 2000, the drift percentage is within the O limit state. After the PGA reaches 0.3 g, the frame structure collapses at a base shear of 3346 kN.The frame displaces 0.290 m, which is 2.93% in drift. The drift percentage exceeds the LS and NC limit states.

The IDA curve starts to soften at approximately 0.2 g by displaying a tangent slope less than the elastic slope.The IDA curve subsequently hardens with a local slope higher than the elastic slope. The structural resurrection phenomenon also occurs in this result. For the case of the six-storey frame is subjected to a near-field effect (Fig. 9), the frame shows its elasticity at a base shear of 1058 kN. The frame displaces 0.054 m, which is 0.27% in drift. According to Vision 2000, the drift percentage is within the O limit state. Then the PGA reaches 0.07 g, the frame structure collapses at a base shear of 1185 kN.The frame displaces 0.355 m, which is 1.79% in drift. The drift percentage exceeds the LS limit state but not the NC. The structure is assumed to experience global collapse because only the earliest failure is regarded.

The IDA curve starts to soften at approximately 0.06 g by displaying a tangent slope less than the elastic but subsequently hardens with a local slope higher than the elastic. The structural resurrection phenomenon also occurs in this result.

Fig. **10** shows the IDA curve for the six-storey frame subjected to a far-field effect. The frame shows its elasticity at a base shear of 1980 kN. The frame displaces 0.105 m, which is 0.53% in drift. According to Vision 2000, the drift percentage is at the O limit state. When the PGA reaches 0.052 g, the frame structure collapses at a base shear of 2366 kN. The frame displaces 0.375 m, which is 1.90% in drift. The drift percentage exceeds the LS limit state but not the NC. The structure is assumed to experience a global collapse because only the earliest failure is regarded.

The structural resurrection phenomenon occurs when a structure returns to a lesser displacement and drift. In this case, the IDA curve starts to soften at approximately 0.04 g by displaying a tangent slope less than the elastic slope.The IDA curve subsequently hardens with a local slope higher than the elastic slope. Meanwhile, in the nine-storey frame is subjected to the near-field effect (Fig. 11), the frame shows its elasticity at a base shear of 1124 kN. The frame displaces 0.044 m, which is 0.15% in drift. According to Vision 2000, the drift percentage is within the O limit state. When the PGA reaches 0.06 g, the frame structure collapses at a base shear of 1481 kN. The frame displaces 0.522 m, which is 1.76% in drift. The drift percentage exceeds the LS limit state but not the NC. As mentioned in the previous paragraph, regarding the earliest failure, the structure is assumed to experience global collapse. The IDA curve starts to soften at approximately 0.052 g by displaying a tangent slope less than the elastic but subsequently hardens with a local slope higher than the elastic. The structural resurrection phenomenon occurs in this result.

A similar behavior is demonstrated by the nine-storey frame under the far-field effect (Fig**.** 12). Elasticity is shown at a base shear of 1161 kN with a 0.040 m displacement and 0.13% drift. When the PGA reaches 0.052 g, the frame structure collapses at a base shear of 2285 kN with a 0.784 m displacement and 2.64% drift. The drift percentage exceeds both the LS and NC limit states. Subsequently, the frame experiences structural resurrection at a PGA of 0.06 g.

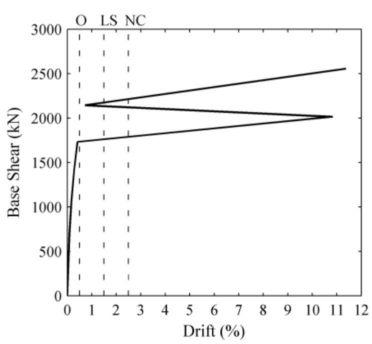

Fig. 7. IDA Curve for the 3-Storey Frame under a Near-Field Effect.

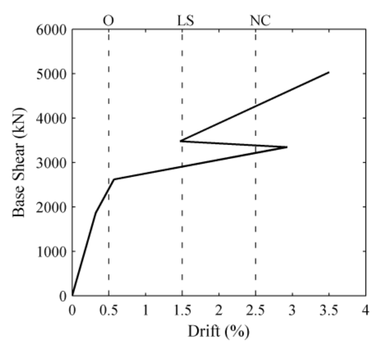

Fig. 8. IDA Curve for the 3-Storey Frame under a Far-Field Effect.

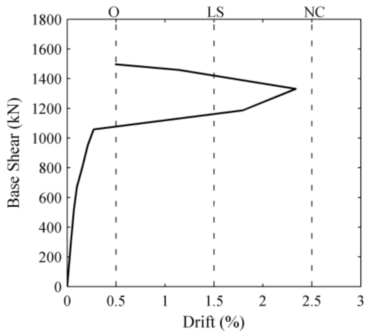

Fig. 9. IDA Curve for the 6-Storey Frame under a Near-Field Effect.

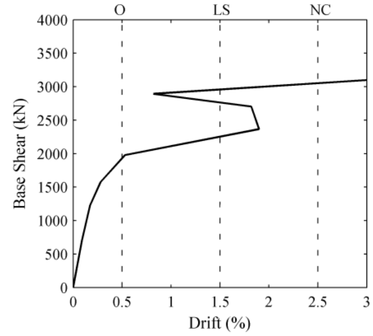

Fig. 10. IDA Curve for the 6-Storey Frame under a Far-Field Effect.

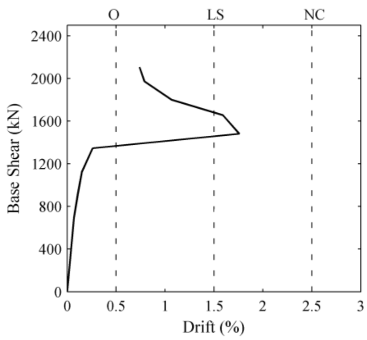

Fig. 11. IDA Curve for the 9-Storey Frame under a Near-Field Effect.

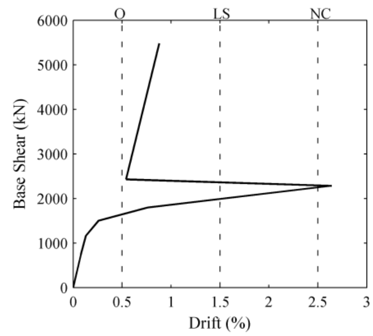

Fig. 12. IDA Curve for the 9-Storey Frame under a Far-Field Effect.

3.3. POA and IDA

IDA curves generally have a higher base shear than POA curves. Therefore, the base shear–drift relationship in the IDA produces higher demand than that in POA. This condition allows engineers to estimate the structural demand in a conservative manner.

In both IDA and POA, MRCFs start to experience global instability before meeting the LS and NC limit states. Thus, the designed MRCFs have poor seismic resistance,which may be due to the following reasons:

a) Deficiencies in structural design:
    - Bracing can produce a laterally stiff structure for a minimum of additional materials. The designed MRCFs are improperly braced, thus resulting in a lack of lateral stiffness.
    - Hinge properties are defined only in the bilinear moment–curvature relationship. Such a relationship can hardly describe the structural behavior but is the minimum requirement in EC8. The trilinear moment–curvature relationship is believed to produce accurate results.
b) Earthquake behavior:
    - The behavior of the ground motion records is unpredictable because the effect is differ for each time step. The structure may suffer from

global collapse even at a low PGA. However, the structure may survive the earthquake at a high PGA if the earthquake behavior at each time step is moderate.

In IDA, the base shear produced by the far-field effect is higher than the base shear produced by the near-field effect. Hence,the far-field effect produces higher demand than the near-field effect. This deduction is valid according to the results obtained in this research. However, such a deduction may not be true for all cases when different ground motion records are selected for the IDA because they have different amplitudes, frequencies, and durations.

As illustrated in Figs. 13-15, POA can approximate IDA only at the elastic stage. After the yield point is exceeded, the POA fails the approximation. IDA produces higher structural demand than POA.

The uniform load pattern performs better in estimation than the inverted triangular load pattern because the POA curves produced by uniform loading follow the IDA curves produced by the near-field effect. The inverted triangular load pattern is less accurate because the POA curves produced are slightly lower than the IDA curves. However, both load patterns are unreliable in predicting the IDA curves produced by the far-field effect.

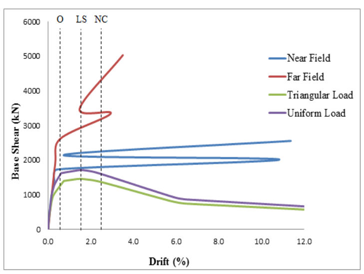

Fig. 13. POA versus IDA of the 3-Storey Frame.

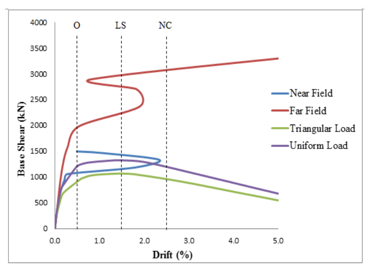

Fig. 14. POA versus IDA of the 6-Storey Frame.

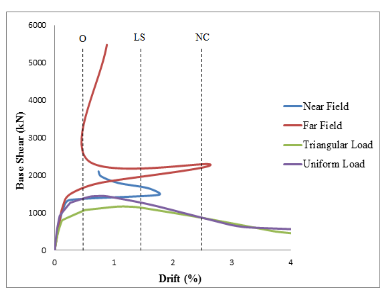

Fig. 15. POA versus IDA of the 9-Storey Frame.

4. Conclusions

This study investigates the damage measure of MRCFs subjected to earthquake excitation with the PBSD limit states. The seismic capacity curves obtained by the POA under two different lateral load patterns were compared with those obtained by IDA. On the basis of the numerical comparisons, the following findings can be concluded:

- The designed structures have low stiffness because all MRCFs fail to meet the NC limit state defined by Vision 2000. This failure might be caused by some deficiencies in design and unpredictable earthquake behavior.
- The base shear–drift relationship in IDA produces higher demand than the base shear–drift relationship in POA.
- In POA, the lateral uniform load pattern produces higher demand than the lateral inverted triangular load pattern.
- In IDA, the far-field effect produces higher demand than the near-field effect. This finding is true according to the results obtained in this research. However, this finding may not be true for all cases when different ground motion records are selected.
- POA approximates IDA accurately at the elastic stage but fails the approximation after the yield point. The uniform load pattern produces a relatively accurate estimation of the IDA curve generated by the near-field effect.

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