Incremental dynamic analysis of underground subway station structure

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Abstract. Incremental dynamic analysis constitutes the basis of seismic performance evaluation and seismic vulnerability assessment. In this paper, a typical two-story three-span subway station structure is selected as an example structure and its incremental dynamic analysis procedure are presented, including selection of ground motion, intensity measure and limit state determination. The incremental dynamic analysis procedure provided in this study can be a basis for further study on seismic design for underground structural systems.

1 Introduction

With the continuous development and utilization of urban underground space, the safety of underground structure is of great importance, especially the structural safety under seismic load. Up to now, many researchers focused on the seismic performance of underground structure [1-5]. Among these studies, seismic fragility analysis based on incremental dynamic analysis (IDA) has been applied to many different kinds of underground structures, such as subway station structure[2], underground cavities[4], tunnel[5]. Through probabilistic approach, the full range of structural dynamic behavior, from elastic to elastic-plastic, until collapse can be considered. However, the implement of IDA for underground structure is still based on surface structure proposed by Vamvatsikos and Cornell in 2002[6] and few study focused on the detail of IDA for underground structure, such as how to choose effective intensity measure, ground motions.

For this purpose, a two-story three-span subway station structure is taken as a typical example to illustrate the detail of IDA, mainly including selection of ground motion, intensity measure and limit state determination. The results obtained herein can be the basis for further study of seismic performance for underground structure.

2 Case study

2.1 Typical subway station structure

The rectangular reinforced concrete box is of 20.9 m wide and 12.37 m high, as shown in Figure 1. The first floor of the station is the lobby floor while the second floor is an island platform. The cross section of all the columns is 0.6m by 1.0m. All column spacing is 8 m. The roof depth of the example structure is 2.9m, the soil condition from the geological investigation report is shown in Table 1.

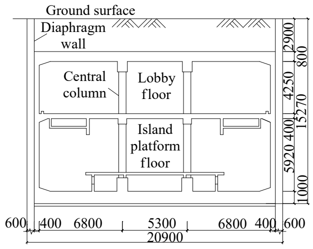

Figure 1. Cross section of the subway station structure.

Table1. Soil parameters of the example structure.

Sequence	Name	Depth (m)	Unit Weight (kN/m^3)	Elastic Modulus (MPa)	Poisson's Ratio	Friction Angle (°)	Cohesion (kPa)
1	Man-made fill	0-1.3	19.0	20.34	0.32	15.0	20.0
2	Isabelline silty clay	1.3-2.4	19.2	20.34	0.32	31.3	9.5
3	Isabelline silty clay	2.4-3.3	18.0	14.00	0.34	33.8	15.1
4	Grey mucky soil	3.3-6.9	17.4	10.85	0.38	28.3	5.3
5	Grey mucky soil	6.9-14.8	16.7	7.39	0.40	24.9	7.2

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6	Grey clay	14.8-16.7	17.4	11.55	0.35	29.7	10.0
7	Dark green clay	16.7-21.1	19.5	24.85	0.29	29.1	31.3
8	Grey silt	21.1-28.0	18.2	32.20	0.29	31.1	2.0
9	Grey clay	28.0-43.0	17.7	15.09	0.33	32.5	8.1
10	Grey clay & Silty sand	43.0-60	18.4	28.70	0.32	28.1	8.0

Table2. Material Parameters of the subway station.

Component	Material					
	Concrete				Steel	
	Elastic Modulus (GPa)	Axial Tensile Strength (MPa)	Axial Compression Strength (MPa)	Possion's Ratio	Elastic Modulus (GPa)	Yield strength (MPa)
Central column	33.5	2.51	29.6	0.2	200	400
Others	31.5	2.20	23.4	0.2	200	400

2.2 Numerical model

The two dimensional finite element model with both the surrounding soil and subway station structure is established according to the general profile mentioned above through the finite element software ABAQUS[7], as shown in Figure 2. The subway station structural model is 1000 m long and 60 m high. To enhance the calculation efficiency, beam element (B21) is adopted for the reinforced concrete structural members here. The material properties of both concrete and steel in the subway station structures are shown in Table 2, which are obtained from the engineering project at Shanghai.

The concrete plastic damage model proposed by Lubliner et al.[8] and Lee and Fenves[9] was adopted. The reinforcement of the two dimensional frame is attained through the rebar command.

The 4-nodes plane strain element (CPE4R) and the quadrilateral plane strain infinite element (CINPE4) were adopted for soil element and the Mohr-Coulomb model was used to simulate the soil's constitutive characteristics. The soil parameters are shown in Table 1. The interface between the soil and the structure is modeled as a frictional surface with a coefficient of friction μ of 0.4 and a friction angle of 22°. There is no cohesion between the structure and the soil.

The boundary conditions of the model are as follows: the horizontal and vertical displacements are fixed at the bottom surface while the top of the structure is free. Infinite elements were applied at the lateral boundaries and the ground motions are imposed at the bottom of the model.

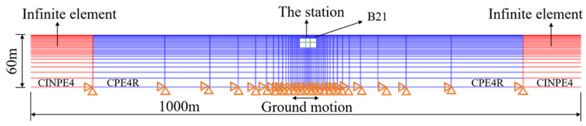


Figure 2. Finite element model.

3 Incremental dynamic analysisprocedure

IDA can take the randomness of seismic ground motion as well as the seismic response under strong ground motion intensity into consideration [6]. In this section, the IDA procedures are discussed detail by detail.

3.1 Selection of ground motion

Generally, 10-20 records are enough to provide sufficient accuracy in the estimation of seismic demands of buildings. Besides, the selected ground motion records should be able to represent the structure's site condition. According to the NEHRP(National Earthquake Hazards Reduction Program),the site condition can be divided into six groups, denoted as A, B, C, D, E, F[10]. The character of the site in this study refers to E group based on the shear wave velocity. Therefore, for IDA of underground structure, a series of twelve ground motion records that belong to a bin of relatively large magnitudes of 7.5-8.0 and near-fault are selected from the Pacific Earthquake Engineering Research Center [11], as illustrated in Table 3.

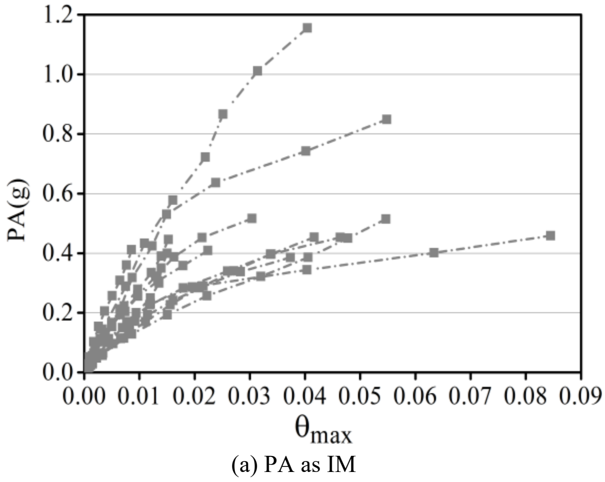
3.2 Selection of intensity measure

Intensity measure (IM) represents the intensity of ground motion, which plays an important role for the accuracy of IDA. In this section, four indexes are selected as IM, including Peak Acceleration (PA), Peak Velocity (PV) at the basement rock and Peak Base Acceleration (PBA), Peak Base Velocity (PBV) at the bottom of subway station structure. The ground motions are scaled with PV as 0.03, 0.05, 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8m/s and the maximum inter-story drift angle (θ_{max}) is chosen as damage measure of IDA. Then, with nonlinear dynamic time-history analysis, the IDA curves can be obtained.

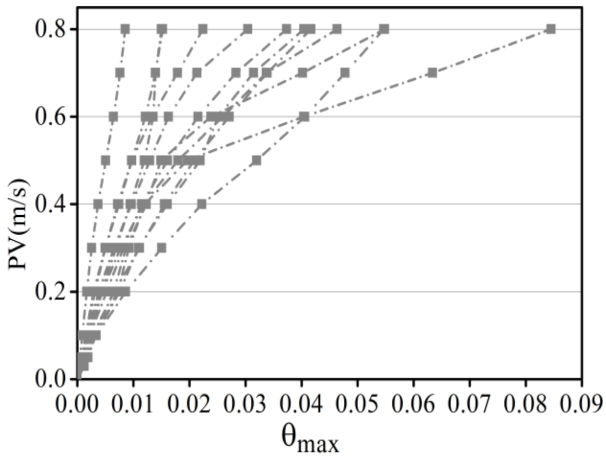
Table3. Twelve ground motion records adopted.

No.	Event	Station	Component	PGA(g)	PGV(cm/s)
M1	Imperial Valley-06, 1979	El Centro Array #3	E03140	0.267	47.97
M2	Imperial Valley-06, 1979	El Centro Array #3	E03230	0.223	43.29
M3	Loma Prieta, 1989	APEEL 2 - Redwood City	A02043	0.274	53.65
M4	Loma Prieta, 1989	APEEL 2 - Redwood City	A02133	0.220	34.12
M5	Loma Prieta, 1989	Foster City - Menhaden Court	MEN270	0.110	21.98
M6	Loma Prieta, 1989	Foster City - Menhaden Court	MEN360	0.119	20.93
M7	Loma Prieta, 1989	Treasure Island	TRI000	0.100	15.59
M8	Loma Prieta, 1989	Treasure Island	TRI090	0.160	33.20
M9	Superstition Hills-02, 1987	Imperial Valley Wildlife Liquefaction Array	IVW090	0.179	31.67
M10	Superstition Hills-02, 1987	Imperial Valley Wildlife Liquefaction Array	IVW360	0.208	36.21
M11	Northridge-01, 1994	Carson - Water St	WAT180	0.091	6.33
M12	Northridge-01, 1994	Carson - Water St	WAT270	0.088	8.32

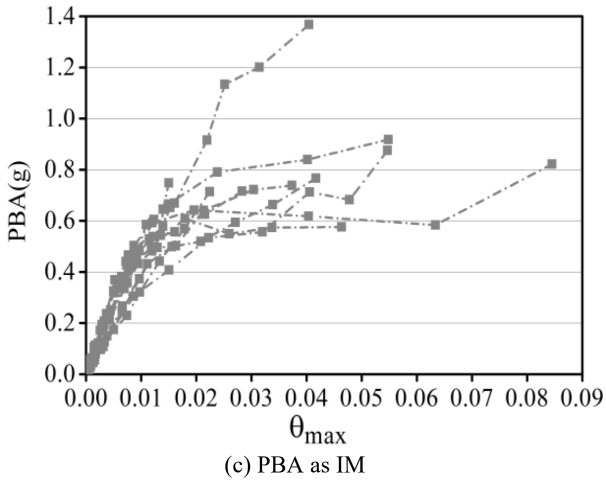
Generally, the basic principle of IM selection is to make the discrepancy of DM under different ground motions as small as possible. The IDA curves with different IMs are shown in Figure 3. To better illustrate the dispersion of curves, we can compare the average values of ln(DM)'s standard deviation, among which the smaller one means lower dispersion and the corresponding IM is more suitable for IDA. Table 4 shows the average values of ln(DM)'s standard deviation of IDA curves with different IMs, respectively. From the statistic in the Table, it can be found that the $\sigma_{\ln(DM)}$'s average value of PBA is smaller than that of the other IMs, while PBV has the largest value among four IMs. Consequently, for seismic performance evaluation of the example subway station structure, PBA is a better IM candidate than the others.



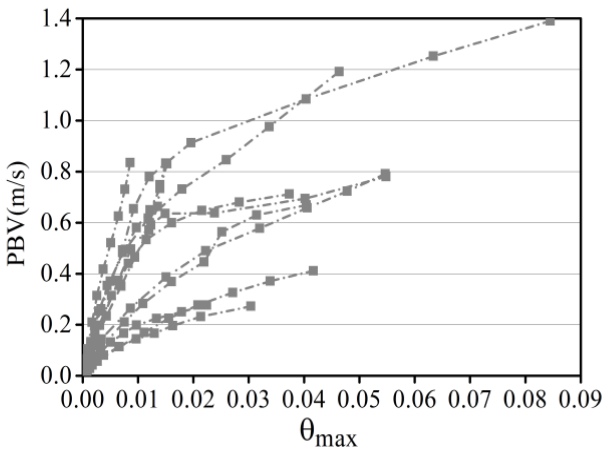
(a) PA as IM



(b) PV as IM



(c) PBA as IM



(d) PBV as IM

Figure 3. IDA curves with different Ims.

Table4. Average value of ln(DM)'s standard deviation.

IM	PA	PV	PBA	PBV
Average value of $\sigma_{\ln(DM)}$	0.485	0.47	0.27	0.505
Ratio of average value to PV	1.03	1	0.57	1.07

3.3 Limit state determination

For surface structure, limit states are divided into two levels: Immediate Occupancy and Collapse Prevention (CP), among which the CP state is defined when the slope of IDA curves decreased to 20% of elastic slope (Ke)[12]. However, when it comes to the IDA of underground structure, the property of IDA curve has great discrepancy due to the restraint of the surrounding soil. To be more clearly, we choose M3 ground motion record, among which Ki means the slope between point i-1 and point i of IDA curve. From the statistics in Table 5, it is shown that the decline of slope is limited and larger than 20% of elastic slope (0.2Ke) all the time.

The result shows that it is not proper for the example subway station structure to define limit state based on the slope decrease of IDA curve. Two approaches are suggested herein. Firstly, define limit state quantitatively based on the design code. Secondly, define limit state quantitatively based on the structural damage distribution under seismic load. These two suggestions have already applied to different kinds of underground structures[2,6,13], but still need further study to make the process of limit state determination more clearly.

Table5. Slope decrease of IDA curve when scale M3.

Slope/Vertical axis	PA	PV
K1(Ke)	30.82091	6027.558
K2	32.63887	6381.01
K3	33.27021	6503.168
K4	32.57325	6369.427
K5	27.06349	5291.005
K6	21.39749	4184.1
K7	22.14286	4329.004
K8	14.20833	2777.778
K9	11.23956	2197.802
K10	11.26652	2202.643
0.2Ke	6.780601	1326.063

4 Conclusion

To conduct IDA of underground structure, effective procedures are provided as follows.

(1) Ground motions for IDA should be able to represent the structure's site condition and 10-20 records are usually enough.

(2) PBA can be selected as the IM for IDA. When the structure type change, its applicability should be verified.

(3) As for the limit state determination, two approaches are suggested and further study is still needed.

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