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.](image)

### Table 1. Soil parameters of the example structure.

| Sequence | Name                | Depth (m) | Unit Weight (kN/m³) | 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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Table 2. Material Parameters of the subway station.

| Component            | 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 $\mu$ 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.

3 Incremental dynamic analysis procedure

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 ($\theta_{max}$) is chosen as damage measure of IDA. Then, with nonlinear dynamic time-history analysis, the IDA curves can be obtained.
Table 3. 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               | TR1000                     | 0.100  | 15.59     |
| M8  | Loma Prieta, 1989         | Treasure Island               | TR1090                     | 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 σ_{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.
Figure 3. IDA curves with different Ims.

Table 4. Average value of ln(DM)'s standard deviation.

| IM       | PA     | PV     | PBA    | PBV    |
|----------|--------|--------|--------|--------|
| Average value of σ_{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 \( K_i \) 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 slope change of IDA curve. 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.

Table 5. Slope decrease of IDA curve when scale M3.

| Slope/Vertical axis | PA    | PV    |
|---------------------|-------|-------|
| \( K_1(Ke) \)       | 30.82091 | 6027.558 |
| \( K_2 \)           | 32.63887 | 6381.01 |
| \( K_3 \)           | 33.27021 | 6503.168 |

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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