Soil Dynamics and Earthquake Engineering 113 (2018) 241–255

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

Experimental verification of current seismic analysis methods of reinforced soil walls

Magdi M. El-Emam

American University of Sharjah, Sharjah, United Arab Emirates

ARTICLE INFO

Keywords:
Reinforced soil walls
Seismic performance
Shaking table
Failure surface
Seismic design

ABSTRACT

The paper investigates the accuracy of two limit equilibrium design approaches used for seismic stability analyses of reinforced soil walls. Responses from a series of reinforced soil wall models, tested on shaking table, are compared to similar responses predicted using both NCMA, and AASHTO/FHWA design guidelines. First, orientation of the soil slip surface measured during subsequent base shaking was compared to the values predicted analytically. Comparison results suggested that the current design methods tend to under predict the size of the soil failure wedge at different input base acceleration amplitudes. Therefore, the current equation proposed to predict the angle of soil slip surface is modified to match measured values. Second, the horizontal and vertical seismic coefficients used in the current seismic analysis methods are compared to values inferred from shaking table accelerometer responses. Results indicated that equations suggested to calculate horizontal acceleration coefficient for design of reinforced soil walls are non-conservative for input base acceleration larger than 0.3 g. Finally, dynamic earth force magnitudes and locations, and dynamic reinforcement force increments measured from shaking table model tests are used to identify sources of conservatism and non-conservatism of the current design methodologies. Different modifications have been suggested to reduce conservativeness or increase the safety of the design guidelines.

1. Introduction

Reinforced soil walls are considered competitive structures over the conventional retaining walls for their cost and performance, especially in areas with active seismicity. Significant volume of research work have directed towards the static performance and design of reinforced soil since its evolution in practice late seventy of the last century [1–5]. In addition, seismic behavior of reinforced soil walls has been subjected to intensive research investigation in the last 20 years [6–16]. Extensive review of the performance of many reinforced soil retaining walls after major earthquakes was found to be satisfactory as reported in literature [17–23]. Nevertheless, cases of failure of some reinforced soil walls have been reported during major earthquakes (e.g. USA [19], El Salvador [21], and Taiwan [20,22]). Some of these failures have been attributed by Ling et al. [20] and Huang et al. [24] to reinforcement rupture due to reinforcement overstressing, and/or excessive lateral deformations. In addition, tension cracks were observed within and/or behind the reinforced soil mass [19] and attributed to the flattening of the internal failure plane under seismic loading [18,25]. This observation is found to be in contradiction with the assumption of constant failure surface at an angle equal to the static failure surface inclination, as assumed by the current design guidelines [26,27]. Collin et al. [17] and Tatsuoka et al. [18] observed that some reinforced soil walls have generated base sliding and facing panel tilting under earthquake loading, which was attributed to the insufficient reinforcement lengths or flattening of the internal failure plane under dynamic loading. It should be noted that all cases reported for performance study have been designed according to M-O design guidelines. Therefore, additional experimental and analytical research was found necessary to identify the degree of conservatism (i.e. sources of good performance) and non-conservatism (i.e. sources of failure or partial damage) of the current design methodologies for reinforced soil retaining walls. In this section, review of the currently implemented seismic analyses procedures of reinforced soil walls, and objectives and scope of this paper are presented.

Seed and Whitman [28] presented the classical Coulomb earth pressure theory that has been modified based on Okabe [29] to calculate dynamic earth forces on rigid conventional retaining walls. The modification consisted of the addition of inertial forces resulted from the movement of the soil under vertical and horizontal ground motion. According to Mononobe-Okabe (M-O) method [29], the total seismic active earth force, P_{AE} imposed by the backfill soil is estimated using the following equation [28]:

E-mail address: melemam@aus.edu.

https://doi.org/10.1016/j.soildyn.2018.06.006
Received 3 July 2017; Received in revised form 4 April 2018; Accepted 6 June 2018
0267-7261/

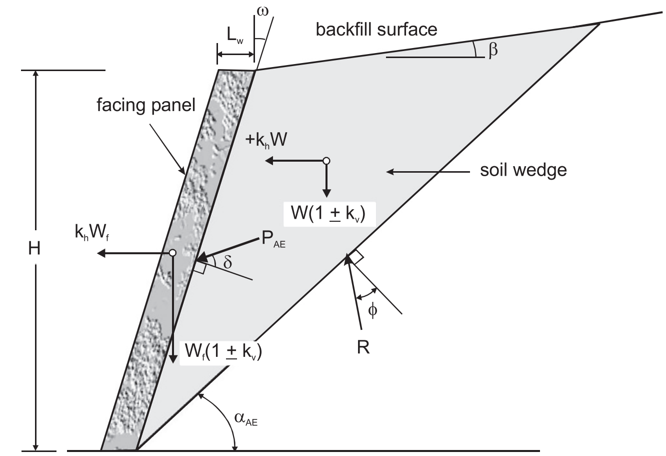

Fig. 1. Forces considered in the Mononobe-Okabe (M-O) analysis, reproduced from [28]. Note: W = weight of the soil wedge; W_f = weight of the facing panel.

$$P_{AE} = \frac{1}{2}(1 \pm k_v)K_{AE}\gamma H^2 \tag{1}$$

Eq. (1) was derived from the force equilibrium of the retained soil wedge shown in Fig. 1 with parameters γ and H represent the unit weight of the soil and the height of the wall, respectively. The total (static + dynamic) earth pressure coefficient, K_{AE}, appears in Eq. (1), can be calculated as [30,31]:

$$K_{AE} = \frac{\cos^2(\phi + \omega - \theta)/\cos\theta\cos^2\omega\cos(\delta - \omega + \theta)}{\left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\cos(\delta-\omega+\theta)\cos(\omega+\beta)}}\right]} \tag{2}$$

where: ϕ = peak soil friction angle; ω = wall face inclination (positive in a clockwise direction from the vertical); δ = back of the wall-soil interface friction angle (or back of the reinforced soil zone-backfill interface friction angle); β = backfill surface inclination angle (from horizontal); and θ is a the seismic inertial angle calculated as $\theta = tan^{-1}[k_h/(1 \pm k_v)]$. Parameters k_h and k_v represent the horizontal and vertical seismic coefficients, respectively, expressed as fractions of the gravitational constant, g.

With the extensive use of reinforced soil retaining walls in seismically active areas, current methods of analysis for conventional earth structures under seismic loading have been extended to cover reinforced soil structures. The analysis methods that have been proposed include pseudo-static rigid body methods that are variants of the original M-O approach. Seismic stability analysis of geosynthetics-reinforced soil structures based on a pseudo-static limit equilibrium method attracted many researchers as reported in literature [32–36]. Most of the previous research work has been developed to verify the proposed seismic analysis and design methods based on shaking table scaled model tests, numerical modelling, and rarely full scale tests and real performance. It was concluded that, the design guidelines and specification for reinforced soil wall structures AASHTO [26], FHWA [27] and NCMA [37] restricted the use of pseudo-static design methods to sites with peak horizontal ground accelerations, a_h less than 0.3 g. However, for more intensive earthquakes (i.e. earthquake with $a_h > 0.3$ g), a displacement analysis should be conducted simultaneously with limit equilibrium analysis. Zarnani [38] reported that, the National Building Code of Canada (NBCC) [39] has recently increased the design peak ground acceleration to 0.6 g at many Canadian cities, especially western parts of British Colombia and Quebec City. Therefore, according to current design guidance documents, pseudo-static design methods cannot be used in these locations. Furthermore, the pseudo-static design approach cannot account for major design parameters such as characteristics of a seismic record (e.g. frequency content, duration, velocity, and situations where resonance is highly possible) [14,40].

The author of this paper strongly believe that the limit equilibrium analysis could be used at any site as long as the site peak ground acceleration, a_g is less than the reinforced soil wall yield acceleration, a_y. This is because wall permanent displacement is expected to take place whenever the earthquake peak ground acceleration, a_g overcomes the available wall yield acceleration a_y, not before that. For such case, the limit equilibrium analysis should be conducted using $a_g = a_y$. El-Emam [40] reported yield acceleration measured on shaking table tests which ranged between 0.3 g to 0.4 g, depending on the model wall design components such as reinforcement stiffness and number of layers, soil stiffness, facing rigidity, inclination angle, height, and toe boundary condition. Critical acceleration is usually predicted from expressions derived from limit equilibrium principles, assuming a factor of safety of one.

El-Emam and Bathurst [41] and El-Emam [42] indicated that, in geosynthetic-reinforced soil-retaining walls with a concrete rigid-facing, the facing column generated additional inertial forces that contributed to peak reinforcement loads. With the exception of the method proposed by NCMA [37], these additional forces are not considered in current design guidelines (e.g. AASHTO [26], FHWA [27]). Furthermore, the influence of the facing type, geometry, inclination, and height on reinforcement loads is not explicitly considered in many current design codes. In this context, Allen and Bathurst [43], demonstrated that the pseudo-static design methods tends to excessively underestimate the reinforcement loads for heavily battered walls.

The current study used results of scaled shaking table tests conducted by El-Emam [40] to verify/modify the current pseudo static seismic design approaches used for reinforced soil walls with rigid facing panel. A total of 14, one-sixth-scale, model reinforced soil retaining walls with full-height rigid panel facings were constructed and tested using the shaking table facility located at the Royal Military College of Canada [14,15,40,41]. The variables between different models included reinforcement length, stiffness, and vertical spacing. In addition, models with different facing stiffness; facing inclination angle; and input base motion characteristics have been constructed and taken to failure using horizontal stepped-amplitude sinusoidal base acceleration records. An advantage of the current shaking table model tests over previous shaking table model tests conducted on reinforced soil walls [9,10,44–49] was the measurement of reinforcement loads and facing toe loads separately at the end of construction and during subsequent base excitation. The experimental shaking table test results used in the current study are briefly presented and discussed. A short overview of the experimental design and instrumentation is included in the current paper for completeness. However, full description of the experimental design, similitude rules, instrumentation techniques, materials, test facility boundary conditions, and interpretation of typical measurement results could be found in El-Emam [40] and El-Emam and Bathurst [14]. The orientation of failure surface observed in shaking table tests is compared with both failure surfaces suggested by AASHTO [26], FHWA [27], and NCMA [37]. Equations, proposed in literature to calculate the horizontal acceleration coefficient (k_h) have been reviewed, compared to experimental results, and modified wherever needed. In addition, earth pressure distribution behind a reinforced soil wall which required for external and internal stability analyses is verified against shaking table results of 14 model wall tests. Finally, the accuracy of M-O variants such as AASHTO [26], FHWA [27], and NCMA [37] design guidelines for calculating the reinforcement forces is investigated and discussed.

2. Objectives of the current study

1. Verify the validity of pseudo-static analysis methods for design of reinforced soil walls, and the suitability of pseudo-static design methods for sites with peak ground acceleration larger than 0.3 g.
2. Investigate the progressive increase of the internal active failure wedge size with increasing magnitude of horizontal acceleration