Soil Dynamics and Earthquake Engineering 80 (2016) 210–224

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

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

Displacement – based parametric study on the seismic response of gravity earth-retaining walls

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ARTICLE INFO

Article history:
Received 26 December 2014
Received in revised form
3 August 2015
Accepted 15 October 2015

Keywords:
Performance-based design
Gravity earth-retaining
Sliding
Non-linear
Time-history analyses
Arias intensity
Newmark's methods
EN 1998-5
FLAC 7.0

ABSTRACT

The essence of performance-based design of gravity earth-retaining structures lies in the estimation of the residual (i.e. permanent) displacements after a seismic event. The accomplishment of this task however can be very complicated due to two interacting phenomena: the coupled sliding and tilting rigid body motion of the wall on an inelastic base and the formation of failure surfaces in the soil backfill. In this study a large number of fully non-linear, time-history analyses of gravity retaining walls (*GRW*) were performed using advanced numerical modelling. Different types of soil parameters and varying wall geometry within a practical range were investigated. The influence of different ground motion parameters was discussed and the results were compared with some of the most common limit equilibrium Newmark's sliding block procedures, including the recommendations by Eurocode 8, Part 5 [20]. Lastly, some recommendations for fast preliminary assessment of the seismic permanent displacements of *GRW* were provided.

1. Introduction

Gravity walls are the oldest type of earth-retaining structures and, although their design under gravity loads is considered simple from an engineering point of view, post-earthquake observations [1,2] and experimental test results [3-8] have shown that the prediction of their residual displacements and predominant failure modes under seismic loading is still a serious challenge for the present analytical and design methods. The issue becomes even more important in the light of the performance-based design concepts. By knowing the residual horizontal displacements and tilting of gravity walls induced by earthquakes, engineers would be able to base their design on prescribed performance levels and on desirable failure patterns like the ductile sliding failure mechanism. What is more, relationships between influential ground motion intensity measures (*IM*) and permanent displacements are valuable for the development of fragility functions, risk assessment and loss estimation.

The approaches for the evaluation of the residual displacements of gravity walls fall into two main categories: displacement-based simplified analytical techniques, which consider systems of rigid bodies displacing along predefined potential failure surfaces and numerical techniques, which account for the non-linear soil properties to predict the magnitude and pattern of stresses and deformations through finite element or finite difference numerical models (stress-deformation analysis methods) [9–12]. Due to their simplicity and ease of implementation the former approaches prevail in the design methods based on allowable displacements like the Newmark sliding-block procedure and its improved variants. Some of the most common ones are listed in Table 1, together with their main assumptions and limitations based on [13] and [14]. The yield acceleration a_y of the potential failure mass, required by the listed analytical procedures, is usually estimated using the Mononobe–Okabe (*M–O*) active soil wedge coupled with the assumption of a constant seismic coefficient (typically 50–70% of the free-field ground acceleration) and an educated guess about the point of application of the soil thrust. Limit equilibrium slope stability computer methods can also be used to determine the yield acceleration.

The limitations listed in the last column in Table 1 have been addressed by many researchers. For example, Matasovic et al. [15]

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http://dx.doi.org/10.1016/j.soildyn.2015.10.012
0267-7261/

Nomenclature

CAV	cumulative absolute velocity
CoV	coefficient of variation
DD	Dobry duration
FS	factor of safety
IM	intensity measure
$M–O$	Mononobe–Okabe
ODF	over-design factor (EN 1997-1)
PE	probability of exceedance
PGA	peak ground acceleration
PGV	peak ground velocity
UD	uniform duration
a_{gR}	reference peak ground acceleration on soil type A
a_h	pseudo-static acceleration
a_{max}	peak ground acceleration
a_y	critical/yielding acceleration
c'	soil effective cohesion (in the Mohr–Coulomb sense)
d	horizontal wall displacement
$d_{perm.}$	residual horizontal displacement of a wall
d_r	allowable horizontal displacement (EN 1998-5)
e	void ratio of the soil
E_d	design value of the effect of an action
f_{max}	maximum frequency
G_0	initial (tangent) shear modulus
G_{wall}	weight of the wall
H	wall height
I_a	Arias intensity
k_h	horizontal seismic coefficient
α	constant related to the geometry of the sliding block
γ_I	importance factor (EN 1997-1)
$\gamma_{R,h}$	partial resistance factor for sliding (EN 1997-1)
$\gamma_{R,v}$	partial factor bearing resistance (EN 1997-1)
δ_d	friction between wall and backfill
δ_f	friction between wall and foundation
Δ_l	grid size
ν	Poisson ratio
μ	mean value
ρ	bulk mass density
σ	standard deviation
σ'_0	mean total stress
φ'	soil effective friction angle (in the Mohr–Coulomb sense)
χ^2	goodness of fit
ψ	dilation angle of the soil
k_v	vertical seismic coefficient
K	bulk modulus
K_o	coefficient for lateral earth pressure
$K_{AE,M\text{-}O}$	earth pressure coefficient with $M–O$ soil wedge
M_w	moment magnitude
P_{AE}	resultant horizontal force
r	reduction coefficient (EN 1998-5)
R_d	design value of the resistance to an action
S	soil factor (EN 1998-1)
$\tan\theta$	tilting of the wall
T	predominant period at which occurs the maximum spectral acceleration at 5% structural damping
v_{max}	peak ground velocity
V_s	shear wave velocity of soil
W	width of wall base
w	settlement of the ground behind the wall

developed a trilinear model for degradation of the yielding acceleration as a function of displacement for geosynthetic surfaces, with which the residual displacements calculated with the Newmark procedure were significantly lower than those obtained with a constant yield acceleration based on residual strength parameters. NCHRP 12-70 [16] suggested that for sloping backfill and high accelerations, when the $M–O$ equation leads to unrealistically large seismic active earth pressure, the limit equilibrium slope stability computer methods might be used instead. In fact, dynamic tests [3–8] have shown that the failure surface in the backfill is almost planar, which was also the conclusion reached by Chen and Liu [17], who used limit analysis theorems and obtained almost planar log-spiral slip surfaces. Another observation reported by the NCHRP 12-70 [16] and based on finite element wave scattering analyses was that the maximum average horizontal acceleration to be used for the evaluation of the earth pressure decreases with the wall height and is a function of the frequency content of the ground motion record. The noncompliant assumption in the conventional Newmark analysis, which treats the unstable mass as rigid, was addressed by Kramer and Smith [18]. The authors developed a two degree-of-freedom analytical model and the results showed that if the fundamental period of the unstable mass was close to the predominant period of the base motion, the conventional Newmark method overpredicts the displacements by up to 100%. This complied with the findings by Gazetas and Uddin [19].

The current design codes are based almost exclusively on the limit equilibrium pseud-static approach. For instance, Chapter 7 of Eurocode 8, Part 5 [20] begins by setting stringent requirements for the method of analysis of retaining structures. According to these requirements, phenomena like non-linear soil behaviour during dynamic soil-structure interaction and the compatibility between soil deformations and the wall displacements should be taken into account. Nevertheless, the model proposed by the code uses simply the $M–O$ active soil wedge with a constant horizontal and vertical acceleration. The horizontal pseudo-static seismic coefficient $k_h=(a_{gR}\cdot\gamma_I\cdot S)/(g\cdot r)$ is calculated by means of a reduction coefficient r (Table 2), which correlates to a certain global factor of safety (FS) and a selected displacement limit (ductility). FS is a combination of partial factors for actions, soil parameters and resistance factors for different limit states, according to EN 1997-1 [21]. The admissible wall displacements d_r are prescribed by EN 1998-5 [20] and "*in the absence of specific studies*" they should be calculated according to Table 2. For example, for European seismicity with $a_{gR}\cdot\gamma_I/g=0.28\,g$ and a soil coefficient $S=1.15$ $d_r=64–97$ mm. This is a rather narrow band of allowable displacements and no further clarifications are given whether these displacement limits refer to serviceability or ultimate limit state and what would be the reduction coefficient r for retaining structures with larger allowable displacements. The seismic coefficient for all wall types varies again within a narrow band $k_h=(0.5\div0.67)a_{gR}\cdot\gamma_I\cdot S/g$ and as a result a few cases of gravity walls satisfy the stability requirement for sliding. For comparison, a Supplementary Guidance [22], released together with guidance for repair of the city of Christchurch (NZ) after the Canterbury sequence in 2010–2011, states that the seismic coefficient for pseudo-static design for ultimate limit state varies between 30% and 70% of the seismic design acceleration according to six different cases of application of the retaining structure, with an allowable displacement of 100–150 mm for $FS=1$. The Italian building code NTC 2008 [23] sets the seismic coefficient at $k_h=(0.2\div0.3)a_{gR}\cdot\gamma_I\cdot S/g$ for an allowable displacement of 200 mm. For lower displacement limits ($d_r=50$ mm), k_h increases to 0.47 [24]. Finally, the guidelines by