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Mitigation of Reverse Faulting Deformation Using a Soil Bentonite Wall: Dimensional Analysis, Parametric study, Design Implications

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Abstract

Recent major seismic events, such as the Chi-Chi (1999) and the Wenchuan (2008) earthquakes, have offered a variety of case histories on the performance of structures subjected to reverse faulting–induced deformation. A novel faulting mitigation method has recently been proposed, introducing a soft deformable wall barrier in order to divert the fault rupture away from the structure. This can be materialized by constructing a thick diaphragm-type soil bentonite wall (SBW) between the structure and the fault rupture path. The paper investigates the key parameters in designing such a SBW, aiming to mitigate the fault rupture hazard on shallow foundations. The paper employs a thoroughly validated finite element analysis methodology to explore the efficiency of a weak *SBW* barrier in protecting slab foundations from large tectonic deformation due to reverse faulting. A dimensional analysis is conducted in order to generalize the validity of the derived conclusions. The dimensionless formulation is then used to conduct a detailed parametric study, exploring the effect of *SBW* thickness w/H, depth H_{SBWl}/H, and shear strength τ_{soil}/τ_{SBW}, as well as the bedrock fault offset h/H, foundation surcharge load $q/\rho g B$, and fault outcrop location s/B. It is shown that the wall thickness, depth, and shear strength should be designed on the basis of the magnitude of the bedrock fault offset, the location of the fault relative to the structure, and the shear strength of the soil. The efficiency of the weak barrier is improved using lower strength and stiffness material compared to the alluvium. A simplified preliminary design methodology is proposed, and presented in the form of a flowchart.

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Keywords
Soil bentonite wall; Near fault ground displacement; Reverse fault; Mitigation; Shallow foundation

1. Introduction

In an earthquake, there are two types of ground displacements as transient dynamic waves and permanent quasi-static offsets on the fault itself [1]. During recent major seismic events (e.g., Kocaeli 1999; Düzce 1999; Chi-Chi 1999; Wenchuan, 2008), the faulting deformation caused considerable damage to structures. One of the best such examples is the 2008 Mw 7.9 Wenchuan earthquake in China, during which surface fault scarps of up to 9 m were observed. **Figure 1** shows an example of a building which collapsed due to the imposed faulting-induced deformation in the area of Bajiaomiao, Hongkou city [2]. To date, research on the quasi-static offset of the fault has mainly focused on: (a) understanding the mechanism by analyzing case histories of structures subject to faulting [3-5], (b) physical modeling [6-13], (c) numerical or analytical studies of fault rupture propagation and its effects on structures [4, 14-21] and (d) proposing mitigation measures against fault rupture hazard [19, 22-26].

Several mitigation strategies can be found in the literature, which can be classified in three different groups: (i) foundation strengthening, aiming to minimize superstructure distress; (ii) measures aiming to diffuse the fault deformation over a wider area; and (iii) measures aiming to divert the fault rupture [9, 22-25]. The first strategy is very straight-forward, and simply requires introducing a rigid raft foundation. Thanks to its rigidity, the latter allows the building to rotate as a rigid body without being distressed. Such a strategy can be effective for both reverse and normal faults [9,22,23], but is difficult to apply for the retrofit of existing critical facilities. Moreover, although such foundation strengthening is efficient in terms of avoidance of collapse, the rigid body rotation of the structures is unavoidable and therefore the post-seismic serviceability of the structure is an unresolved issue. The second strategy employs a ductile compacted earth fill beneath the foundation, aiming to diffuse the faulting deformation [9]. However, this calls for a sufficiently thick and ductile earth fill to be installed underneath the foundation, requiring replacement of a substantial soil mass. Obviously, such a strategy can only be applied to new structures.

A key advantage of the third strategy is that it can readily be applied to new *and* existing structures. Instead of foundation strengthening, a diaphragm wall is introduced aiming to divert the fault rupture [24,25]. The basic concept of implementation of a stiff wall barrier has been introduced by Oettle & Bray [25], aiming to offer protection against normal faults. Despite its efficiency, such a mitigation technique requires good knowledge of the location of the fault rupture, which is not always feasible. Therefore, a mischaracterization of the fault location may reduce the efficiency of such a scheme. The "opposite" concept of a *weak* wall barrier has been proposed by Fadaee et al. [24], aiming to offer protection against reverse faults. Such a wall barrier, softer and weaker than the surrounding soil acts as an "attractor" of plastic deformation, diverting the rupture away from the foundation-structure system. Such a scheme offers adequate protection in terms of avoidance of structural damage, but also substantial reduction of permanent rotation of the structure, thus solving the problem of post-seismic serviceability.

Besides introducing the concept, Fadaee et al. [24] investigated the effect of foundation location and superstructure dead load on the effectiveness of the SBW, focusing on a specific example. The present paper conducts a dimensional analysis and introduces non-dimensional parameters related to the SBW, aiming to derive deeper insights on the design procedure and generalize the validity of the derived conclusions. An extensive parametric study is performed, employing a thoroughly validated numerical analysis methodology. More specifically, the study investigates the key design parameters of a SBW, which include: (a) the SBW thickness *w,* in function of the fault offset *h* and the foundation surcharge load *q*; (b) the SBW depth H_{SBW}; and (c) the ratio of soil to wall shear strength τ_{soil}/τ_{SBW}. Based on the results of the parametric study, an illustrative flowchart is presented, to be used for preliminary design of such a SBW.

2. Problem definition and methodology

The aim of this paper is to investigate the effect of influential parameters on the design of a SBW for protecting a foundation against reverse faulting. As schematically illustrated in **Fig. 2**, the studied problem refers to a thrust fault of dip angle α, producing upward displacement of vertical amplitude *h*, propagating through a uniform soil deposit of thickness *H*. A stiff raft foundation of width B carrying a surcharge load q, is positioned at distance s from the theoretical point of rupture outcropping in the free field. The foundation is protected by a SBW of height H, thickness w_{SBW} and shear strength τ_{SBW}, which is positioned at distance L from the left edge of the foundation. The validity of the derived conclusions is strengthened through a combination of experimental and numerical work. The effectiveness of the concept, as well as the validity of the numerical modelling, are confirmed by reduced–scale physical model testing. The validated model is subsequently used to conduct an extensive parametric study. Various wall thicknesses and depths, as well as shear strength ratios (τ_{soil}/τ_{SBW}) are examined in the parametric study. The difference between shear strength ratio of SBW and surrounding sand plays an important role in the absorption of the induced deformation by fault rupture. In this study, the shear strength of the sand and SBW are calculated using the equation $\tau_{Sand} = \sigma_v tan(\varphi)$ and $Su/\sigma_v' = 0.25$, respectively.

2.1 Physical modelling

Physical modelling is employed to conduct a parametric study of the problem. The results of tests that were conducted at the International Institute of Earthquake Engineering and Seismology of Iran (IIEES) are presented in this paper. A split-box was designed and constructed to simulate quasi-static fault rupture propagation through soil and its interaction with foundation–structure systems. The apparatus consists of a fixed and a movable part, which can move downwards or upwards to simulate normal or reverse faulting, respectively. The dip angle of the fault is adjustable from 45^{o} to 90^{o}. The two sides of the apparatus are equipped with Plexiglas windows, allowing observation of the deformed specimen and computation of the deformation field through image analysis. Firoozkooh No.161 sand was used in the tests, which is a uniform fine-grained material, having a uniformity coefficient $C_u = (d_{30} / d_{10})$ of 1.3. **Figure 3** shows grain size distribution curve of the Firoozkooh sand as well as the view of fault rupture simulation box and its dimensions. The dimension of IIEES apparatus is (length x width x height) 1.8 m x 0.5 m x 0.8 m.

Scale effects are indeed critical in small scale experiments. In this regard, Wood investigated the scaling rules in 1g and centrifuge physical tests [30]. Also Bray used the model to prototype properties in the small scale 1g experiments [8]. Although the main scaling rules should be taken into account, considering all requirements in physical models would be hardly applicable. In the conducted physical model test, the shear strength of the soil and wall are scaled down according to the scaling laws proposed by Wood [30]. Given the capacity of the split box, a scale N = 100 was selected for the tests. The sand layer was prepared by dry air pluviation. In order to achieve the desired relative density D_r (80% in the specific tests series) the height of the sand hopper, the aperture of the sleeve, and its velocity are appropriately adjusted. The layering of the sand takes place in layers of approximately 5 cm. After each 5 cm sand layer, dyed blue sand is poured tangent to the transparent windows of the box, in order to create a pattern to capture and identify the propagation of the fault rupture through the soil specimen.

A clay mixture was used to model the SBW, consisting of kaolinite and sodium montmorillonite at a 3:1 ratio and water for SBW shear strength adjustment. The characteristics of the SBW are mainly dependent on the status of in-situ stresses, as well as the construction method of the wall. The SBW is constructed by pouring a mixture of soil and bentonite into the excavated trenches without any compression. Thanks to its lower stiffness comparing to the surrounding soil, the stress in the soil-bentonite mixture is lower than the geostatic stress condition for a long time after construction. The accomplished in-situ experimental tests indicated that the soil-bentonite mixture possess very low stiffness [27]. Since the cohesion is not largely dependent on the stress field, small-scale (1g) testing of cohesive materials can be considered realistic, provided that the fundamental scaling rules are applied: $S_u^{(reality)} = S_u^{(experiment)} \times N$ [8,30]. In the absence of appropriate experimental tests, if the mixture contains more than 35% plastic fine-grained, it is possible to consider $Su/\sigma v'$ = 0.19 [26]. The shear strength of SBW during fault rupture propagation can be considered conservatively as the consolidated undrained clay. After reaching the desired moisture content of 45%, the clay mixture is consolidated using odeometer apparatus. Further, a number of direct shear tests with vertical stresses ranging from 15.8 kPa to 200 kPa are carried out. Regarding the results derived from direct shear tests and the empirical correlations between Su and PI, the undrained shear strength of the utilized clay mixture was estimated as S_u/σ_v' = 0.25, leading to an average $S_u \approx 0.3$ kPa. A secant Young's modulus $E/S_u = 300$ was considered appropriate [24,25,26]. **Table 1** indicates the basic characteristics of SBW. Naturally the properties of the SBW are time dependent. Initially, the clay mixture is at a liquid state during backfill placement. In the first few months after construction it consolidates, reaching its final shear strength.

Two steel plates are placed in the fault box at the desired locations in order to install the wall in 1g models. Then sand is poured to the required depth. After that the clay slurry is poured between two plates and the rest of the box filled with sand. At the end, two plates are extracted carefully [24].

In our tests, two different measuring approaches were employed: (a) a digital inclination meter was used to measure the vertical displacement and the rotation of the foundation; and (b) image processing applying the PIV method, to compute the strain and displacement field across the image domain including soil deformation, rupture path, separation of the foundation, and surface profile [24]. The deformed soil specimen was captured every 2 mm of imposed base offset by a high-resolution (8 MP) digital camera. All images were rectified using the derived optical parameters of the whole system (camera property, Plexiglas, etc). Note that after rectifying and scaling, all lengths are real and can be measured directly from the picture. Subsequently, the relative motion of sand particles in time and space is derived by computing the optical flow between each pair of consecutive rectified images of the sequence. The absolute position of sand particles can be computed by tracing the relative motion back to the initial image. Similarly, after computing the position of two points on the foundation it is possible to calculate foundation rotation and translation [28].

2.2 Numerical modelling

Finite element (FE) modelling has been shown to be capable of efficiently reproducing fault rupture propagation in the free field [4,29], and its interaction with surface and embedded foundations [18]. The analysis is conducted employing the commercial FE code ABAQUS (2011). The soil is modelled with quadrilateral continuum elements of dimension $d_{FE} = 0.5$ m to achieve a reasonably refined mesh under plane strain conditions. Following the findings of previous studies, an elastoplastic constitutive model with Mohr-Coulomb failure criterion and isotropic strain softening was adopted [4]. Pre-yield behaviour is modelled as linear elastic, with a secant modulus $G_s = \tau_y / \gamma_y$ increasing linearly with depth.

Strain softening is introduced by reducing the mobilized friction angle ϕ_{mob} and the mobilized dilation angle ψ_{mob} with the increase of plastic octahedral shear strain. The displacements δ_{xy} and δ_{xf} are related to the yielding and residual state of the soil, and can be derived from direct shear test. These parameters are subsequently used to calibrate the soil constitutive model [4]. During the Quaker project, genuine Class A predictions were conducted, before conducted centrifuge model tests at the University of Dundee [15]. Despite the unavoidable distortions, the results have been shown to compare very well with the experiments, and this increase our confidence on the validity of the employed method. In addition, it has been shown to be capable of simulating fault rupture propagation through sand and its interaction with surface and caisson foundations with reasonable accuracy [15,18,33]. Furthermore, in this paper the numerical simulations are shown to compare very well, with the conducted small-scale experiments. The slab foundation is simulated using linear elastic beam elements, and is connected to the soil through special contact elements which are infinitely stiff in compression but tensionless allowing detachment of the foundation. **Figure 4** shows the schematic view of the numerical model as well as the assumed boundary conditions.

2.3 Calibration of soil parameters

Two loose and dense sands were selected for our analyses. Two samples of No.161 Fkooh sand with relative density of 55% (loose) and 80% (dense) were constructed. Direct shear tests were carried out to measure peak and post-peak strength characteristics for both sand densities, and for a normal stress range from 100 kPa to 400 kPa. The measured fundamental properties are summarized in **Table 2**. While the physical model tests were conducted using dense D_r = 80% sand, the calibrated loose D_r = 55% sand was subsequently utilized for the parametric numerical study. The stiffness of the sand is a function of confining pressure, therefore a linear distribution of the Young's modulus is assumed. The strain softening model is used to reproduce the results of the direct shear tests, in order to demonstrate its capability to reproduce the measured soil behaviour as illustrated in **Fig. 5**, the numerical prediction of the laboratory direct shear tests is quite satisfactory.

3. The effectiveness of SBW using experimental tests

In this section, the key results of protected and unprotected 1g physical model tests are compared in order to demonstrate the effectiveness of the proposed method. Unless otherwise stated, all of the results are presented in prototype scale.

3.1 SBW Interaction with a surface foundation

Two experiments were conducted in order to investigate the efficiency of the SBW in protecting the foundation. A first test (Test.1RF) was conducted, modeling a B = 15 m shallow foundation at distance s = 5 m (or s/B = 0.3 in dimensionless terms) from the unperturbed fault rupture, without any mitigation measures. To focus on the effectiveness of the *SBW*, a relatively light uniform surcharge load q = 30 KPa was used in the experiments (so that the observed fault rupture diversion could solely be attributed to the presence of the *SBW*). A second test (Test.2RWF) was conducted, in which a *SBW* was installed at distance 8 m from the left edge of the foundation, aiming to intercept the rupture path. The deformed soil model with superimposed displacement vectors (as computed through image processing) is compared with the deformed FE mesh, for bedrock fault offset h = 2 m for both protected and unprotected cases (**Fig. 6a** and **6b**). A separation gap between the bottom of the foundation and the ground soil is clear in both physical and numerical models as highlighted in Fig. 6a. As shown in **Fig. 6c**, the foundation rotation in unprotected condition reaches almost 6º for h = 2 m (or h/H = 10% in dimensionless terms). Such a rotation is well beyond acceptable serviceability limits. The evolution of foundation rotation with bedrock fault offset h can be broadly categorized in three distinct phases of response (**Fig. 6d**). In the first phase "A", which corresponds to relatively small bedrock fault offsets (h < 0.8 m), all of the imposed faulting–induced deformation is absorbed by *SBW.* As a result, the foundation rotation ϑ is practically negligible. In the second

Commented [P1]: In response to comment 1 phase “B” (0.8 m ≤ *h* < 1.3 m), the *SBW* is approaching the limits of its compression capacity, and a limited amount of the imposed tectonic compression finds its way to the soil which is located on the footwall side (i.e., to the right). Consequently, the foundation starts rotating; but, since the deformation is still quasi-elastic, the rate of increase of ϑ is relatively small. Note that, in zones A and B (up to bedrock fault offset of h/H = 7%), the foundation rotation does not exceed the commonly acceptable angular distortion (1/300 to 1/500). Finally, in phase “C” (*h* ≥ 1.3 m), the *SBW* cannot absorb any more compression, as the horizontal component of the fault offset becomes comparable to its thickness *w* = 3 m. This renders the deformation mechanism kinematically inadmissible, leading to the development of the aforementioned secondary rupture. The second rupture path outcrops near the left edge of the foundation, unavoidably leading to an appreciable increase of the rotation ϑ. Nevertheless, even for *h* = 2 m, the *SBW* is still quite effective in reducing ϑ from about 6^o (**Fig. 6**) to less than 2.5^o.

3.2 Influence of wall thickness

Another experiment (Test.3RWF) was performed in order to explore the effect of the thickness of the SBW. The latter was installed at exactly the same distance from the left edge of the foundation, with the only difference being the wall thickness, which was reduced to 2 m. The numerical prediction is compared to the experimental results in **Fig. 7**. Images of the deformed physical model with superimposed displacement vectors are compared to the FE deformed mesh for two different bedrock fault offsets: *h* = 1 m (**Fig. 7a**) and *h* = 2 m (**Fig. 7b**). For *h* = 1 m, the *SBW* absorbs most of the faulting–induced compression, but a fault branch finally develops to the right of the wall, emerging close to the left edge of the foundation. As a result, the foundation rotation ϑ is increased to roughly 1^o. Further increase of the fault offset to *h* = 2 m leads to the development of a third fault branch, which is initiating at a depth of about 14 m. This rupture propagates upwards, outcropping just to the left of the foundation, leading to a non-negligible increase of its rotation. The evolution of foundation rotation with bedrock fault offset *h* is evident in **Fig. 7c**. The phases of response are similar to those previously described (**Fig. 6**) but the thresholds are smaller due to the reduced thickness of the SBW. Comparing **Figs. 6d** and **7c**, which are related to the rotation of the protected foundation with 2 m and 3 m

SBW thickness, respectively, it is concluded that the reduction of the thickness of the SBW leads to a decrease of its capacity to reduce foundation rotation. For instance, while the rotation of the protected foundation with a 3m thickness SBW is less than 0.5 degrees (for h = 1 m), reducing its thickness to 2 m leads to a substantial increases of the rotation to 1.5 degrees.

4. Dimensional analyses

Although the parametric analysis presented herein is undertaken for an H = 20 m soil deposit and a B = 10 m foundation, the key results and conclusions are of more general validity. On the basis of dimensional analysis principles [30,31], there are some non-dimensional parameters for fault rupture problems [4,6,8,32]. The bedrock offset h and the vertical displacement Δy can be normalised with soil thickness H, as h/H and $\Delta y /H$. Similarly, the surcharge load q and the foundation bending moment M can be written in non-dimensional form as $q/\rho gB$ and M/qB^2 respectively [19]. In this paper, the new non-dimensional parameters are introduced related to the soil bentonite wall. The SBW width w, its depth H_{SBW}, and its distance from the foundation L can be normalized with soil thickness H, yielding the dimensionless quantities w/H, H_{SBW}/H and L/H, respectively. The ratio between the shear strength of the soil deposit and that of the SBW also plays an important role in the absorption of fault rupture. To express this parameter in dimensionless form, the ratio of the undrained shear strength of the soil bentonite wall to that of the surrounding medium is defined as τ_{soil}/τ_{SBW}. In terms of stiffness, the dimensionless ratio is defined as E_{SBW}/E_{soil}. However, scale effects tend to complicate the problem further, rendering the normalization not strictly accurate for all dimensionless parameters. In order to verify its validity, from an engineering point-of-view, a parametric study is conducted exploring the response for two different cases (Models A and B) having different dimensions, but sharing the same dimensionless parameters, as summarized in Table 3. The dimensions of the two cases examined are summarized in **Fig. 8a**. The results are compared in terms of vertical displacement profile at the ground surface (**Fig. 8b**; foundation rotation for different fault offsets (**Fig. 8c**; normalized foundation contact pressures p/q for h/H = 5% (**Fig.8d**; and normalized foundation bending moments M/qB^2 (**Fig. 8e**). The comparisons verify the validity of the dimensional analysis, as the results are practically identical in dimensionless terms.

5. Parametric numerical study

The efficiency of the proposed mitigation technique is a function of a number of parameters, related to the magnitude of the imposed fault offset, the width and surcharge load of the foundation, soil properties (strength and stiffness), and the dimensions and properties of the *SBW*. The effect of foundation location (relative to the free-field fault outcrop, expressed through distance s, and the surcharge load q of the superstructure on the efficiency of a specific SBW of w/H = 0.15 (w = 3 m) thickness and H_{SBW}/H = 0.75 (H = 15 m) depth have already been examined in a previous publication [24]. Taking advantage of the previously discussed dimensional analysis an extensive parametric study is conducted herein in order to derive deeper insights on the following dimensionless parameters:

(a) *SBW* thickness: w/H = 0.05, 0.1, and 0.15 (i.e, w = 1, 2, and 3 m, for H = 20 m);

(b) *SBW* depth: H_{SBW}/H = 0.5, 0.625, and 0.75 (i.e., H_{SBW} = 10, 12.5, and 15 m, for H = 20 m); and

(c) shear strength ratio τ_{soil}/τ_{SBW}: dense and loose sand, as summarized in **Table 2**.

The comparisons are performed for different bedrock fault offsets h/H = 1.5%, 3%, and 5%, also considering a range of dimensionless surcharge loads $q/\rho gB$ = 0.1, 0.2, and 0.4 (i.e., q = 20 kPa, 40 kPa, and 80 kPa, representative for 2 to 8-storey buildings). Since the exact location of an active fault cannot be predicted with accuracy, the parametric analysis is conducted for various dimensionless locations s/B = 0.1 to 1.1 (i.e., 1 to 11 m for a B = 10 m foundation). For s/B = 0.1, the free-field fault rupture would outcrop at a distance of $0.1B$ from the left edge of the foundation, which means that the foundation is mostly lying on the footwall. For s/B = 1.1, the free-field fault rupture would outcrop at a distance 1.1B from the left edge of the foundation (or 0.1B from its right edge), which means that the foundation is entirely on the hanging wall. The results are compared in terms of foundation rotation ϑ and dimensionless bending moment M/qB^2, which are both considered to be important indexes of foundation performance. Even if the stressing of the foundation is acceptable, or can be accommodated for by foundation strengthening, the rotation ϑ is crucial for the serviceability of the structure. In the following sections, the key results and conclusions of the parametric study are presented and discussed.

5.1 The effect of *SBW* thickness

The required *SBW* thickness is a function of the imposed bedrock fault offset *h/H* and the surcharge load *q/ρgB*. While the latter is determined by the characteristic of the structure, the design fault offset should ideally be estimated on the basis of a probabilistic fault displacement hazard analysis (PFDHA). In this section, to focus on the effects of the *SBW* thickness *w/H,* a B/H=0.5 (*B* = 10 m) foundation and *SBW* thickness *w/H* varying from 0.05 to 0.15 (from 1 to 3 m) are considered. The analysis is conducted for *s/B =* 0.1 *to* 1.1. The influences of thickness are inspected in terms of fault offset and foundation surcharge load.

Fault offset

The required SBW thickness is directly related to the design fault offset. **Figure 9** compares the performance of different *SBW* alternatives with the unprotected base case (**Fig. 9a**) in terms of FE deformed mesh (for *h/H* = 5%) for a foundation of width *B/H* = 0.5 positioned at *s/B* = 0.5. In the case of a relatively thin *w/H* = 0.05 (i.e., *w* = 1 m) *SBW* (**Fig. 9b**), although a substantial part of the imposed compressive deformation is absorbed by the SBW and the main fault rupture is effectively diverted, a secondary fault branch develops, propagating towards the foundation. By increasing the thickness of the *SBW* to *w/H* = 0.10 (i.e., *w* = 2 m)*,* the secondary rupture still develops but does not reach the surface (**Fig. 9c**). With an even thicker *w/H* = 0.15 (i.e., *w* = 3 m) *SBW* (**Fig. 9d**), it disappears completely as all of the imposed tectonic deformation is effectively absorbed by the weak barrier, with the foundation being almost completely unaffected.

These findings are further corroborated in **Fig. 10,** which compares the foundation rotation ϑ as a function of the dimensionless bedrock fault offset *h/H* for all cases examined (always using the unprotected case as a reference). The thin *w/H* = 0.05 *SBW* can be seen to be effective for *h/H* ≤ 1.5% (the differences in ϑ are quite minor). For larger bedrock fault offset *h/H*, the *w* = 0.05*SBW* reaches its absorbing capacity and a sharp increase of ϑ is observed. In exactly the same manner, up to *h/H* = 3% the *w/H* = 0.1 *SBW* is sufficient. For even larger fault offsets (*h/H* > 3%), only the *w/H* = 0.15 *SBW* seems to be adequate. Hence, a range of applicability can be defined on the basis of the thickness dimensionless thickness *w/H* of the *SBW*. **Figure 11** depicts the rotation of the foundation for two different fault offsets *h/H* = 3% and 5%, for all fault break locations examined, confirming the aforementioned conclusions. While for fault offset *h/H* ≤ 3%, the *w/H* = 0.10 *SBW* can be appropriate (**Fig. 11a**), for larger values up to *h/H* = 5%, the *w/H* = 0.15 *SBW* is required (**Fig. 11b**). Since the unprotected foundation experiences more rotation when fault rupture outcrops in the middle (i.e. 0.3< *s/B*< 0.9), the existence of SBW in this cases could result in more remarkable reduction in rotation.

Surcharge load

Heavily-loaded foundations have been shown to be able to divert the fault rupture, being subjected to relatively lower levels of distress. In such cases, the performance of the unprotected foundations may be satisfactory in terms of flexural distress [18]. As a result, the SBW may be less efficient in terms of reducing their flexural distress. However, this is not always the case, and foundation rotation *ϑ* may even become larger in some cases [19]. Therefore, even for heavily loaded foundations, mitigation through a *SBW* is still necessary.

The effect of dimensionless surcharge load *q/ρgB* ranging from 0.1 to 0.4 on foundation rotation is explored for different dimensionless *SBW* widths *w/H* ranging from 0.05to 0.15, also varying the positioning of the foundation *s/B* = 0.1 to 1.1 . Comparing **Fig. 11b** with **Fig. 12**, it becomes evident that the increase of the dimensionless surcharge load *q/ρgB*, leads to an amelioration of the efficiency of the SBW, diminishing the difference between the case of *w/H* = 0.10 and 0.15. While for a lightly loaded q/ρgB = 0.1 foundation, a SBW of *w/H* = 0.15 thickness is necessary, for more heavily loaded foundations a thinner w/H = 0.10 foundation may also lead offer adequate protection against the imposed tectonic deformation. Increasing the magnitude of surcharge load from 0.1 to 0.4 changes the range of *s/B* in which the SBW is most effective. For instance in the case of *q/ρgB* = 0.4 (**Fig. 12b**), SBW is more effective when fault rupture outcrops in the right side of the foundation (i.e. 0.7< *s/B*< 1.1).

5.2 The effect of SBW depth

In order to be efficient in protecting the foundation, the weak SBW barrier has to be deep enough to intercept the fault rupture, diverting its path and absorbing the tectonic deformation. The amount of such diversion, and hence the efficiency of the mitigation technique is a function of geometry: wall depth), fault dip, and outcrop location. If the SBW is not deep enough, it will not intersect the rupture path, and the fault rupture will be unaffected. On the other hand, the cost of the mitigation increases with depth. Hence, from an engineering point of view, it is necessary to optimize the depth of the SBW in order to achieve an economical SBW and adequately efficient solution. To illustrate the effect of SBW depth, a parametric study is conducted varying H_{SBW}/H from 0.5 to 0.75. **Figure 13** summarizes the results comparing the performance to the unprotected case for s/B = 0.1 to 1.1. As expected, the deeper H_{SBW}/H = 0.75 SBW offers the best performance in all cases. The performance is still acceptable for H_{SBW}/H = 0.625, but the shallower SBW of H_{SBW}/H = 0.50 is efficient only for $s/B \leq 0.6$.

5.3 The effect of soil strength

To explore the effect of soil strength an example comparison is performed considering dense and loose sand. The difference between shear strength and elasticity modulus of the soil deposit and *SBW* plays an important role in the absorption of the tectonic deformation. Since the mitigation technique is relying on the capability of the SBW to act as an "attractor" of plastic deformation, its efficiency should increase with the contrast of its stiffness and strength relative to that of the surrounding soil. **Figure 14** compares the response in terms of deformed mesh and foundation rotation. The efficiency of the SBW is markedly superior for dense sand (**Fig. 14a**), as most of the imposed deformation is absorbed by the weaker material of the SBW barrier. The rotation ϑ of the protected foundation is reduced by roughly 93% compared to the unprotected case (**Fig. 14b**). The SBW is still efficient in loose sand, but due to the smaller contrast in terms of strength and stiffness, the rotation is decreased by 65%.

Figure 15 compares the performance in terms of envelopes of dimensionless foundation bending moment for all fault locations (s/B = 0.1 to 0.9) and for h/H = 0 to 5%, for a *SBW* of thickness w/H = 0.15 and depth H_{SBW}/H = 0.75 in loose (**Fig. 14a**) and dense sand (**Fig. 14b**). As previously, the results are also compared to the unprotected benchmark case. As expected, the foundation distress is increased with soil stiffness, being substantially higher for dense sand. The efficiency of the mitigation technique is more evident for dense sand, due to the previously discussed larger contrast in terms of stiffness and strength. Still though, the best foundation performance is observed for the SBW-protected foundation in loose sand, with the bending moment being almost unaffected by the imposed tectonic deformation.

6. Discussion on design issues

Although it is theoretically feasible to conduct in-situ testing to locate an underlying fault, the prediction of its exact location is practically impossible in practice. Moreover, there are uncertainties regarding the prediction of the dip angle of the fault. Hence, a range of possible values should be used in design, both with respect to the dip angle and the location of the fault. If such a range can be determined with reasonable accuracy, based on seismo-tectonic and geological studies, a single SBW barrier between the foundation and the fault rupture (i.e., towards the hanging-wall side of the structure) can be sufficient. But if the available data is limited (both regarding the location and the dip angle of the fault), a second SBW barrier at the other side of the foundation (i.e., on the footwall side) may be required.

Among other factors, the necessary depth of the *SBW* is a function of the depth of the soil deposit, and two different cases are considered: (a) shallow soil deposit, up to 30 m depth; and (b) medium to deep soil deposits. In the first case, it is possible to construct a *SBW* barrier reaching bedrock, in order to achieve optimum performance of the mitigation measures, regardless of the fault dip. The 30 m depth is a reasonable limit, as up to this depth the *SBW* can be constructed using standard diaphragm wall machinery. At this point, it should be noted that this mitigation technique cannot be applied to very shallow soil deposits, and there is a minimum depth required for fault rupture diversion. In the second case (for medium to deep soil deposits), the required bedrock offset for outcropping of the fault rupture at the ground surface is larger. The probability of fault outcropping is reduced with the increase of soil depth, and the tectonic deformation may be completely absorbed by the soil, diminishing the need for mitigation measures. Still though, the distortion at the ground surface may still be substantial, leading to non-negligible stressing of the foundation, calling for a detailed parametric numerical analysis to design the *SBW*.

Figure 16 summarizes the design procedure of a *SBW* for mitigation of the fault rupture hazard. In this procedure the rotation of the unprotected foundation under tectonic displacement is a function of its surcharge load (q), soil properties (E and τ_{sand}) and the magnitude of bedrock fault offset (h). The first step of the analysis is to compute the performance envelopes (e.g., bending moments and rotation) for all possible fault locations s/B. If the magnitude of foundation bending moment and rotation does not exceed the allowable values (in absolute terms), no counter-measures are required and the design can be finalized. Otherwise, mitigation measures are necessary and it may be appropriate to protect the foundation employing the proposed method. An initial assumption of *SBW* properties and geometry (depth and thickness) is necessary (which can be based on the results of this study), followed by re-computation of performance envelopes for all possible fault locations s/B. If the computed values do not exceed the allowable values, the design can be finalized. If not, a deeper and/or thicker and/or softer *SBW* is necessary, and the procedure must be repeated until satisfying the conditions. Although the parametric study is conducted for h/H=5%, the proposed procedure can be extended for other possible bedrock offsets. The SBW reduces the rotation of the foundation substantially, provided that its thickness is sufficient. To derive the range of the outcropping locations in which the SBW is most effective, it is necessary to use the design procedure presented in **Fig. 16**. Meanwhile, In order to overcome the uncertainties related to the fault location, a comprehensive parametric study (**Fig. 16**) for the entire range of probable s/B is required.

7. Conclusion and limitation

The paper has applied a thoroughly validated FE methodology to explore the efficiency of a weak *SBW* barrier in protecting slab foundations from large tectonic deformation due to reverse faulting. A detailed parametric study has been conducted in order to derive deeper insights on the performance of the proposed mitigation technique and some practical recommendations on the contemplated design procedure. A dimensional analysis has been conducted in order to generalize the validity of the derived conclusions. Taking advantage of the dimensionless formulation, a parametric study has been conducted exploring the effect of *SBW* thickness *w/H*, depth H_{SBW}/H, and shear strength τ_{soil}/τ_{SBW}, as well as the bedrock fault offset *h/H*, the foundation surcharge load *q/ρgB*, and the fault outcrop location *s/B*. The key conclusions can be summarized as follows:

a) The necessary wall thickness *w/H* is a function of the design bedrock fault offset *h/H*. For a large *h/H*, a thicker *SBW* is required in order to absorb the imposed tectonic deformation. If the wall thickness is not sufficient, one or more secondary fault branches may develop, reducing the efficiency of the mitigation and leading to increased foundation rotation ϑ and bending moment M/qB^2.

b) The performance is ameliorated with the increase of the surcharge load *q/ρgB*, and mitigation measures may not be necessary in terms of foundation distress. Still though, a *SBW* may be needed to reduce the foundation rotation ϑ at acceptable serviceability levels. The necessary thickness *w/H* of the SBW is reduced with the increase of *q/ρgB*.

c) The stiffness and strength of the *SBW*, expressed through the contrast ratio τ_{soil}/τ_{SBW} is shown to affect its efficiency substantially. The efficiency of the weak barrier is increased with the decrease of its strength and stiffness: a weaker and more compressible *SBW* is more efficient in diverting the fault rupture and absorbing the imposed tectonic deformation.

d) The necessary SBW depth is mainly a function of geometry and should be determined through a parametric analysis of all possible fault locations *s/B* and for an appropriate range of fault dip angles (if the exact value is unknown). If the wall depth is not sufficient, the fault rupture path may not be diverted completely and mitigation may be insufficient.

e) A flowchart is presented, summarizing the methodology to design mitigation measures using a *SBW*. In a first step, the performance envelopes (rotation and stressing) is computed for the unprotected foundation, considering all possible fault locations. If the performance is not acceptable, an initial SBW configuration (thickness, depth, shear strength and position) is assumed, and the performance envelopes are recomputed and evaluated via the acceptable values in terms of ultimate and serviceability limit states. The procedure is repeated until satisfactory foundation performance is achieved.

g) The main concept of this paper is to modify the rupture path by a weak wall. Although the accomplished parametric study in this paper is conducted for sandy material, the concept can be applicable to other soil materials, based on the principle of minimum energy. Still though the conclusions are valid for the cases examined, and further study is necessary to prove the validity for different soil types.

h) The thickness of SBW should be reasonable in the practices for useful mitigation and design consideration. If higher magnitudes of bedrock fault offset is estimated on the basis of a PFDHA, a thick SBW may be required. However, its thickness can be increased by constructing two or more walls. This paper is preliminary steps of offering an efficient mitigation measure, and admittedly more work is need to solve practical and construction issues.

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Table1. Basic properties of SBW

ρ	1.4 ton/m^3
PI	80
Moisture content	45%
E/Su	300
Su/σv	0.25

Table2. Summary of material properties

	Loose	Dense
E (first layer, kPa)	2500	12000
E (in last layer, kPa)	14000	50000
φ_p	32	42
φ_{res}	30	32
Ψ_p	2	12
Ψ_{res}	1	1
Υ_f	0.2	0.165
ρ (ton/m^3)	1.65	1.8

Table3. Dimensional analysis parameters

dimensionless quantity	magnitude
w/H	0.15
H_{SBW}/H	0.75
L/H	0.25
E_{soil}/E_{SBW}	5.55
τ_{soil}/τ_{SBW}	2.95
h/H	0.05
$q/\rho gB$	0.1
B/H	0.5
α	45
s/B	1.1