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THE SOCIETY FOR EARTHQUAKE AND
CIVIL ENGINEERING DYNAMICS

NEWSLETTER

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In this issue

Ian G Smith, Chairman of SECED, introduces an open letter to the ICE Director of Engineering Policy concerning the future of SECED.

Seismic shift for SECED?

Dear Member,

I hope you are enjoying our popular Newsletter and I write briefly just to draw your attention to a potential problem that seems to be looming concerning a part of the future funding of SECED. The letter on the following two pages to the Director of Engineering Policy and Innovation at the Institution of Civil Engineers (ICE) is self-explanatory and I hope that he will be able to convince ICE Council of the need to maintain ICE's support to SECED at least at its current level. Other ICE Associated Societies have similar funding problems.

I remain hopeful that this matter will be resolved but we are continuing to make plans for reducing key items of our current expenditure; the principal item being the ICE services we pay for. This may result in us looking for alternative meeting venues as we move into Spring meetings next year.

Assuring you of our best endeavours and that we'll keep you briefed on any key developments, as necessary.

Ian G Smith
Chairman

The Society for Earthquake and Civil Engineering Dynamics

The Institution of Civil Engineers
One Great George Street, Westminster
London, SW1P 3AA
Telephone: +44 (0)20 7222 7722
Fax: +44 (0)20 7222 7500
e-mail: seced@ice.org.uk

Mr Andrew Gooding, BSc CEng FIMechE FCIBSE
Director Engineering Policy and Innovation
The Institution of Civil Engineers
1 Great George Street
Westminster
LONDON, SW1P 3AA

02 October 2009

Ref : IGS/02/10/09/1

Dear Andrew,

The Society for Earthquake and Civil Engineering Dynamics (SECED)
Service Level Agreement – Balance of Contributions ICE/SECED

I refer to our exchange of e-mails this week regarding the above and confirm my understanding of the current situation:

- The Service Level Agreement (SLA) between SECED and the ICE defines mutually agreed services and financial arrangements between the parties and is renewed annually every year;
- Currently, and for some time now, the ICE has been generous in recognising the support Associated Societies, like SECED, require in order to function effectively;
- A key feature of this support has been the ICE's contribution under the SLA of 75% of SECED's operating costs related to the ICE services;
- Effective from January 2010, ICE Council has determined that the ICE's contribution to SECED under the proposed 2010 Service Level Agreement will reduce from 75% to 50%. This will effectively double the cost to SECED of using the ICE services, though I note that the room hire charges element will be reduced by 11% at the same time.

SECED is amongst the more active Associated Societies and this action, along with the termination of interest payments on our assets will see our rapid demise if we do not take action now; we are already starting to draw on our reserves.

Following our Committee Meeting this week, SECED would draw your attention to the following observations:

- SECED considers that it should have a close association with the ICE, since its subject matter crosses the whole civil engineering spectrum. Nevertheless, SECED has lots of other natural homes and is not beholden to the ICE;
- SECED considers that it brings value to the ICE many times any ICE perceived support the ICE gives SECED, enhancing ICE reputation and position as a learned body;

.... Cont'd

The Society for Earthquake and Civil Engineering Dynamics

The Institution of Civil Engineers
One Great George Street, Westminster
London, SW1P 3AA
Telephone: +44 (0)20 7222 7722
Fax: +44 (0)20 7222 7500
e-mail: seced@ice.org.uk

- SECED contributes many hours of work by leading engineering practitioners without charge to the ICE;
- SECED will continue to tighten its finances in the current economic climate and will not pass them onto its members and guests, and trusts that the ICE has the same aspirations;
- SECED considers that the ICE's behaviour in relation to the SLA is not proportionate in merely seeking further revenue for its services from the societies, especially in what should only be a short-term dip in the ICE's long-term revenue stream;
- In the absence of any discussion on this matter by Council with the Associated Societies, SECED is looking at its options to safeguard its future;
- If such options result in SECED and other Associated Societies leaving the ICE, SECED consider that the ICE will effectively become a mere financial/conference service body, no longer fulfilling its Royal Charter. In particular, it will severely damage its declared objective from the Royal Charter, that *"The object for which the Institution is constituted is to foster and promote the art and science of Civil Engineering."*

In conclusion, I would request that you ask Council to reconsider its proposal to change the funding arrangements within the SLA and, through this, enable the societies to focus on the excellent work started a couple of year ago by the ICE to promote the standing of the Associated Societies role within the ICE.

Yours sincerely,

Ian

Ian G Smith
Chairman (2008-2010)
The Society for Earthquake and Civil Engineering Dynamics (SECED)
Chief Engineer, Design and Engineering, ATKINS

cc Jean Venables, OBE, ICE President, via SECED Secretary
Jade Donovan, SECED Secretary (file)
Associated Society Chairs, via SECED Secretary
SECED Committee
SECED Members, via Newsletter

Seismic earth pressures – a technical note

Merrick Taylor
Arup Geotechnics, London

This brief technical note summarises the key methods in codes and the literature and how they are applied. It is a synthesis of papers presented by the author last year at the New Zealand Geotechnical Symposium [1] [2] and recent experience on projects in Europe. The focus is on pseudo-static earth pressures used in typical day-to-day engineering practice, where the transient dynamic load of an earthquake is simply represented by an additional static pressure or force resultant on the structure being designed. The use of more advanced methods to consider the dynamic response is only discussed briefly where understanding the dynamic soil-structure interaction may be important, such as deriving the effective design inertia.

Seismic design

Philosophy

For static design, the aim of code guidance is to ensure that catastrophic collapse or failure is avoided (Ultimate Limit State or ULS), but also to ensure that the performance of the structure under the applied load will not be disappointing in terms of wall movements – be they settlements or deflections (Serviceability Limit State or SLS). For the seismic case we wish to check essentially the same limit states for a "design" earthquake. The design earthquake will have been specified by national or industry code, or a special study to determine the acceptable level of hazard for the structure. This increasingly considers different levels of seismic hazard to ensure the performance at frequent earthquakes is not disappointing (analogous to an SLS check) as well as ensuring rare earthquakes do not lead to failure (a ULS check). Here "failure" may be defined in terms of unacceptable wall displacements rather than a catastrophic failure of the wall. A clear understanding of the required performance of the structure in terms of displacements is advisable prior to undertaking design. Modern codes (e.g. Eurocode 8 (EC8) [3]) may implement this philosophy implicitly by assuming performance during frequent events will be sufficient for normal structures if the ULS check performed for rare events is satisfied with a given safety margin. Facilities supporting hazardous materials and processes (e.g. LNG, Nuclear) tend to have more explicit requirements for the respective hazard levels – the latter will have usually

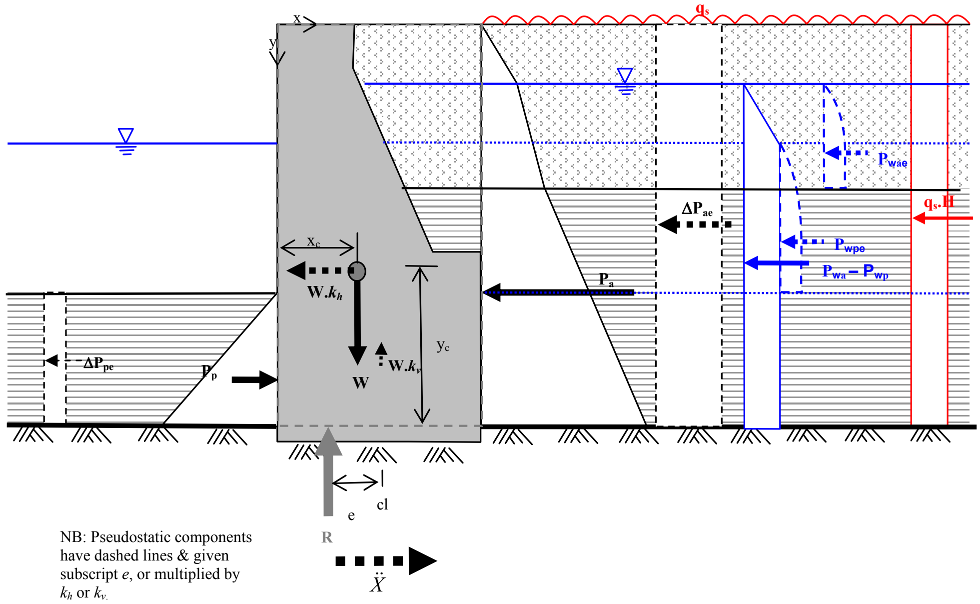
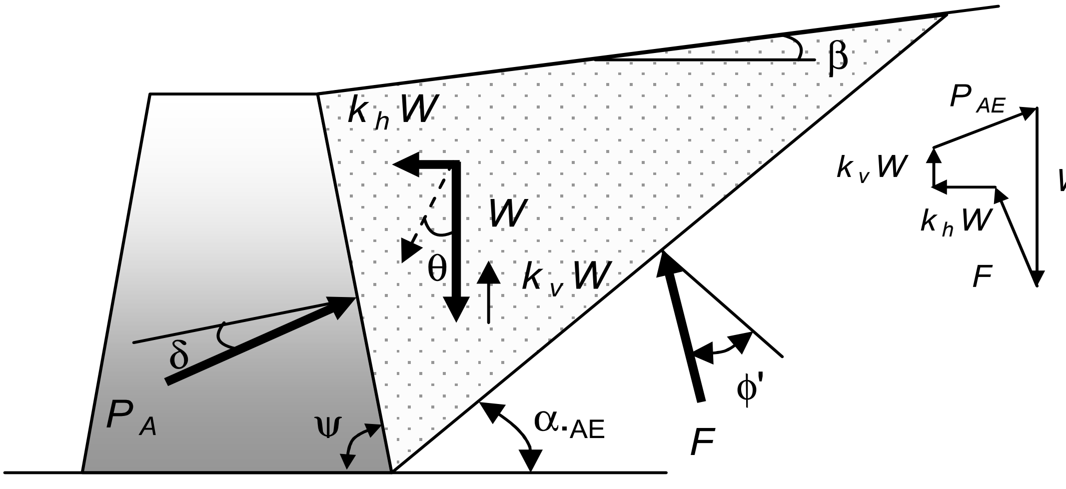


Figure 1. Typical loading diagram for a gravity quay wall under seismic loading, assuming active earth pressure conditions occur at the ULS.



k_h / k_v = horizontal/vertical seismic coefficient

$$\theta = \arctan\left(\frac{\gamma_{ratio}}{1-r_u}\frac{k_h}{1 \pm k_v}\right)$$

(a)
$$K_{AE} = \frac{\sin^2(\psi+\phi-\theta)}{\cos(\theta)\sin^2(\psi)\sin(\psi-\theta-\delta)(1+X)^2}$$

where $X = 0$ if $\beta > \phi - \theta$, otherwise:
$$X = \left(\frac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\sin(\psi-\theta-\delta)\sin(\psi+\beta)}\right)^{0.5}$$

(b)
$$K_{PE} = \frac{\sin^2(\psi+\phi-\theta)}{\cos(\theta)\sin^2(\psi)\sin(\psi+\theta)(1-Y)^2}$$

where $Y = 0$ if $\theta > \beta + \phi$, otherwise:
$$Y = \left(\frac{\sin(\phi)\sin(\phi+\beta-\theta)}{\sin(\psi+\beta)\sin(\psi+\theta)}\right)^{0.5}$$

$\gamma_{ratio} \approx 1.6$ for dynamically pervious backfill, 2.0 for impervious backfill, 1.0 for dry. Ref. Matsuzawa et al. [7]

Figure 2. Mononobe–Okabe dynamic earth pressure coeffcient calculation; (a) active and (b) passive limit conditions. Coefficient *K* is the ratio of lateral to vertical effective stress at the wall-backfill interface. NB: The passive condition has no wall friction *δ* considered, after EN 1998 [3].

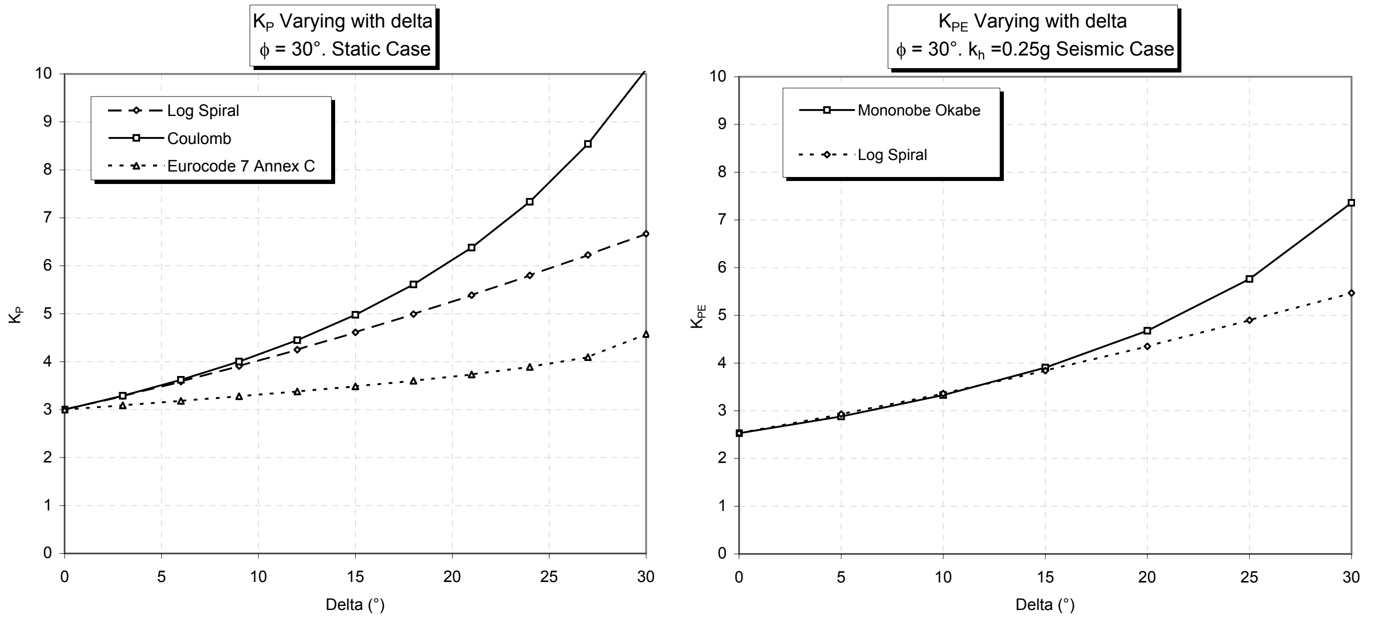
been derived from a special study carried out by Engineering Seismologists for these types of structures.

Methodology

The design methodology has been expounded by Steedman [4] in a paper on seismic design of retaining structures. In addition, useful guidance, particularly in relation to the performance based design framework may be obtained from PIANC [5] and to a lesser extent from EC8. A typical loading diagram for a quay wall under pseudostatic loading is shown in Figure 1. Inertia loads on the retaining structure should also be considered in addition to inertia load on the soil and water.

Dynamic earth pressure theories

The commonly cited methods to assess seismic earth pressures (or "dynamic earth pressures") in codes and texts make basic assumptions about how the wall and soil interact, or together referred to as the "wall-soil-system". These fall into the two extremes of system response: perfectly rigid or no deflection or displacement; and walls free to displace and/or deflect until minimum (active) earth pressures occur. Selection of the appropriate method will depend on the structure being considered, and the definition of the ULS for the particular structure. The bulk of this paper is geared to describe these two basic methods, their



(a) (b)

Figure 3. "Coulomb error" with large angles of wall friction. (a) Static passive earth pressures and (b) Seismic passive earth pressures.

assumptions and limitations. It also presents alternative approaches and guidance from the literature where they are required.

Rigid walls free to displace to active-earth conditions

Mononobe-Okabe method

The dynamic earth pressures at limit equilibrium may be estimated using the classic "Mononobe-Okabe" (M-O) [6] [7] earth pressure method, where the applied inertia results in rotation of the principal stress (angle θ in Figure 2), enlarging the Coulomb active wedge (or decreasing for the passive case) at limiting equilibrium. The method enables calculation of the lateral earth pressure coefficient K, which is a ratio of horizontal to vertical earth pressure at the wall-backfill interface. In general, M-O provides a relatively good estimate (cf. model testing), provided the assumptions are met; chiefly that the wall is able to deflect or displace away from the soil to the minimum limit condition – thus mobilising full shear on the failure plane; secondly, the soil is rigid such that the acceleration applied is uniform; and thirdly, the soil is cohesionless and dry [8]. An important point is that the original method does not make any claim to where the point of action of the resultant force should be applied to the wall, an important consideration for assessing overturning stability.

These assumptions and limitations affect the applicability of the M-O method and raise a number of issues that will be addressed to some extent within the remainder of the paper. These include:

- the "Coulomb error" for passive earth pressures with wall friction applied;
- walls that do not displace sufficiently to form an active wedge;
- the appropriate design acceleration coefficient to apply to the wall backfill;
- the point of action of dynamic thrust;
- cohesive soils; and
- water pressure effects.

The longevity of the M-O method is due in part to its simplicity but also its adaptability as various modifications have been suggested by researchers over subsequent years and eventually adopted in code guidance. It is also however much abused and is often applied to situations which it was never intended – such as rigid walls, tied back and stiff cantilever embedded walls. Care should therefore be taken in its adoption to avoid gross errors.

Passive earth pressure error

An important caveat with M-O for passive pressures with wall friction ratio $\delta/\phi > ½$ is that an unconservative error develops, inherited from Coulomb's passive earth pressure equation which assumes a linear failure plane – in reality it is curved [10]. Because of this, EC8 ignores wall friction entirely. An alternative to remove this slight conservatism for both static and seismic case is to adopt a log-spiral shaped failure plane [11], refer Figure 3. Note that passive earth pressure requires a significant amount of wall deflection to be mobilised, which may well exceed the ULS criteria for the wall. Reference to typical wall movements required to mobilise the active and passive earth pressure conditions is provided in Eurocode 7 (EC7) [12], and NAVFAC [13]. A factor should be applied to limit the mobilisation of passive earth pressure for most design situations where the wall is embedded in soil.

Large design inertia

The M-O method becomes unstable when the sum of interia angle θ and backfill slope angle β exceed the angle of shearing resistance of the soil, ϕ'. In this case, the square-root term on the denominator of the equation becomes complex, and cannot be solved. Matsuzawa et al. [9] proposed a simplification to avoid this problem which is adopted by EC8 and is included in Figure 2. However, this results in very large active wedge angles, which may be unrealistic in reality. The recommended approach to avoid this problem is to consider the critical acceleration at which the wall will begin to displace (i.e. factor of safety = 1), and carry out a performance based assessment of the retaining structure (more discussion is provided on this aspect in a subsequent section on displacement estimation). An alternative approach is to consider the modification of the M-O method by Koseki et al. [14], which considers the punctuated development of multiple wedges:

1. The initial active wedge occurs under static conditions with peak soil strength (ϕ'_{peak}) and no inertia applied (k_h = 0). Shearing continues along the pre-defined wedge until residual strengths (ϕ'_{res}) are developed on the failure plane. The earth pressure increases, and this is used for static design.
2. If the active wedge has not developed under static conditions, it may do so when moderate earthquake inertia is applied to the same wedge geometry as 1. They recommend a value of 0.2g for typical design as the critical value at which the wedge forms (using M-O approach). Again, lateral earth pressures are considered using a wedge geometry based on peak strengths, but with a reduction to residual strengths.
3. The peak earthquake inertia above this critical value causes a secondary larger wedge to develop with peak strengths. This will cause larger earth pressures to act on the wall.

The approach has been referenced by the ISO draft code on performance based design in earthquake geotechnical engineering [15], and Japanese Geocode 21 [16]. The main advantage over M-O is that consideration is made to the development of active wedge prior to the earthquake occurring, and consideration of the effect of strain softening

Figure 4: Damage to quay wall at Derince, Leman industrial facility. Kocaeli, Turkey Earthquake of 17/8/99. Image courtesy Earthquake Engineering Field Investigation Team (EEFIT), UK [24].

from peak to residual along the pre-defined wedge. For large events, the peak strength of the soil may be considered in the formation of a new active wedge, which assists in avoiding the problems with M-O and large inertia.

Point of action of dynamic thrust

From studies in the literature three interacting components have been identified to affect the point of action of the dynamic earth pressure resultant ("dynamic thrust"). These are:

- Ground motion frequency
- Wall-soil system relative flexibility
- Global movement of the wall-soil system.

Seed and Whitman [8] divided the M-O earth pressure resultant (P_{AE}) into separate static (P_A) and incremental dynamic (ΔP_{AE}) components $P_{AE} = P_A + \Delta P_{AE}$, and recommended based on model testing, that the incremental dynamic component be considered to have a point of action at 0.6H measured from the base (H = wall height). Often this advice was simplified by adopting an inverted triangle for the dynamic earth pressure profile. Steedman and Zeng [17] in a pseudodynamic analysis of the active wedge showed the point of action was a function of the height of the wall, the shear wave velocity of the backfill, and the period of the ground motion; essentially showing that the point of action of ΔP_{AE} is dependent on the proportion of inertia that affects the upper third of the assumed active wedge, where the bulk of the mass resides. Thus the 0.6H of Seed and Whitman was at best an upper-bound and would be conservative for design purposes, particularly in a performance-based framework.

Veletsos & Younan's [18] dynamic analysis of fixed base walls considers a visco-elastic medium without a pre-defined wedge. Their work provides the point of action of the dynamic thrust for varying wall-soil system flexibilities from Wood's ~0.6H for perfectly rigid wall [19], down to less than ⅓H for very flexible walls. Richards et al. [20] investigated the mode of wall movement and showed that rotation about the base caused the point of action to drop to the lower ⅓, whilst for a translation mode it was at 0.5H, and for rotation about the upper portion of the wall 0.67H.

It is perhaps in light of this work from the preceding decade that EC8 is the first modern code to depart from the 0.6H or inverted triangle convention and recommends applying the point of action of the dynamic component at mid-height or 0.33H if free to rotate about the toe.

Cohesive soils

A number of methods have been adopted to consider cohesive soils in the literature. Whitman [21] refers to some with a degree of scepticism. Recently Anderson et al. [22] consider soils with significant c' and ϕ' using limit equilibrium slope stability software to determine the dynamic earth pressure coefficient K_{AE}, and provide charts for practical use. For the undrained case ($\phi' = 0$), the use of the same approach or a trial wedge method may be adopted to determine the dynamic earth pressure. The Japanese Ports and Harbours design manual [23] provides an equation for determining the undrained dynamic earth pressure. Consideration should be given to reduction in the shear strength due to cyclic loading and generation of excess pore pressures concurrent with the application of peak dynamic loading. An alternative is to consider an effective stress based approach using a modified M-O method, which is discussed in the subsequent section.

Dynamic water pressures

For many retaining structures in terrestrial environments, walls are designed with drainage to ensure the build up of static water pressures does not occur, however this is not possible for walls permanently below the water table – be they basements, underground structures, or marine structures such as quay walls. Here the presence of a permanent static water table has a significant effect on wall stability. During earthquakes this has three important effects:

1. The weight vector in the M-O active wedge (refer Figure 1) is almost halved due to buoyancy, thereby greatly increasing angle of the weight vector θ. This effect was reported by Matsuzawa et al. [9], who noted that for free draining conditions during cyclic loading (referred to as "dynamically pervious", such that excess pore pressure generation under cyclic loading is minimal), this factors the effect of horizontal inertia by approximately 1.6 (dry unit weight/unit weight of water). For "dynamically constrained" conditions (i.e. fine grained deposits that during dynamic loading will result in essentially undrained behaviour), the factor is around 2.0 to 2.2, depending on the ratio of saturated unit weight to unit weight of water. It is recommended that care is taken to derive the saturated and dry weights from soil phase relationships so that this ratio is correctly estimated as the magnitude of the factor can have a significant effect on the estimated dynamic pressures.
2. If the soil remains dynamically pervious – e.g. open or coarse granular fills that allow the free flow of water between grains, dynamic water pressures should also be considered. The method of Westergaard [25], developed for free water bodies such as dam reservoirs, is adapted for the presence of soil grains – once again reference is made to [9] for guidance on this assessment; it is also a useful guide as to whether the material will behave as "dynamically pervious" or "dynamically constrained". EC8 ignores the relative effect of soil grains on the dynamic water pressures in the backfill, and it is true that the effect is generally relatively minor.
3. If the soil is dynamically constrained, a degree of excess pore pressures will be generated during cyclic loading, as with each successive cycle of loading pore pressures generated due to volumetric changes on shearing can

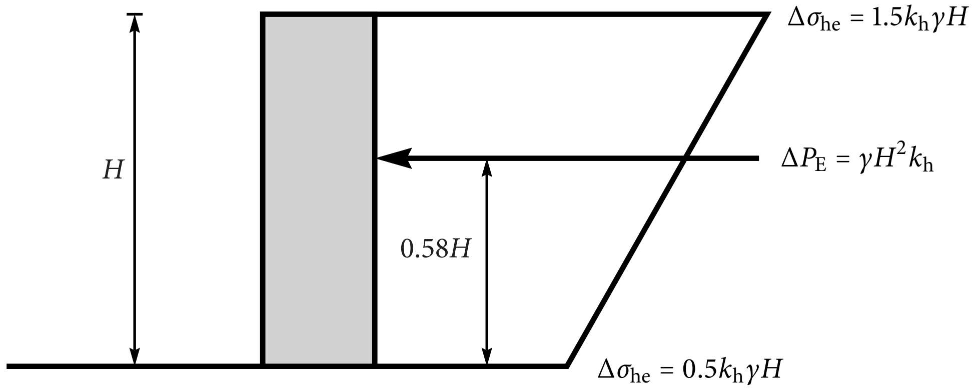


Figure 5. Dynamic earth pressure profile on rigid walls, after Matthewson et al. [26].

not dissipate in time before the subsequent cycle of shearing occurs. The effective stress path will thus progressively work its way towards the failure envelope, or phrased in total stress terms, the shear strength degrades progressively with each subsequent cycle. The main concern in this instance is for soils whose steady state shear strength – that is, the strength of the soil after large shear strains have taken place – is less than the in-situ strength, resulting in complete collapse of the deposit (flow liquefaction). Most catastrophic failures of quay walls during earthquakes are due to this problem occurring in hydraulic fills (e.g. Port Island, Kobe 1995, Derince Port, Turkey 1999 – refer Figure 4). Preventing this problem should be the first priority of the designer, principally through the use of ground improvement techniques such as stone/vibro columns. The condition where generation of excess pore pressures build up progressively but do not necessarily lead to liquefaction, should also be considered as strength is lost from the deposit, and dynamic earth pressures will be larger. One mitigating effect is that earthquake energy is consumed in order to shear the deposit, and by the time significant pore pressures are generated, the peak loading cycles may have already occurred. The guidance to consider excess pore pressure ratio r_u in the calculation of angle θ is provided by [10] as a modification to reference [9]. The estimation of r_u for a given deposit and level of earthquake shaking is beyond the scope of this paper.

Elastically constrained rigid walls

A rigid wall will typically not deflect sufficiently to develop an active or passive failure wedge. Thus most codes recommend Wood's [19] solution (e.g. Eurocode 8 (EC8) [3]) for an elastically constrained rigid wall:

$$\Delta P_E = \gamma H^2 k_h F_p \, , \tag{1}$$

where F_p is a dimensionless thrust factor and a function of the stiffness of the system, and H the height of the wall. For typical soils F_p is approximately unity, and is what EC8 assumes. This may be thought of as a uniform inertia k_h applied to a rigid block of soil of dimensions $H \times H$. The point of action for this force resultant was determined to be $0.58H$, sometimes simplified to $0.6H$. To determine the dynamic earth pressure profile, the recommendation of Matthewson et al. [26] is often adopted, where the dynamic earth pressure decreases linearly from the top of the wall at $1.5k_h\gamma H$ to the base at $0.5k_h\gamma H$ – refer Figure 5. The static earth pressure component is based on *at-rest* pressures or compaction pressures where the wall has backfill present. By mere observation it is clear the dynamic load will be significantly greater than the M-O method, and for many engineering problems will be very conservative.

Fixed base flexible walls

It may be appreciated that there are many cases when neither of the base assumptions recommended by codes (i.e. M-O and Wood) are strictly applicable, in which case further analysis beyond the basic code methods may be required. Veletsos and Younan [18] have shown Equation (1) to be valid for rigidly elastic wall-soil conditions, but if the wall-backfill, and/or wall-base are more flexible, the dynamic earth pressures can be considerably less. They provide tabulated F_p factors for varying soil-wall and soil-foundation flexibilities, demonstrating Wood's solution for rigid systems, through to typical wall flexibilities ($F_p \approx 0.5$) and down to very flexible systems where $F_p \approx$ M-O values of ΔK_{AE} (i.e. K_{AE} - K_A, where the latter is based on the Coulomb equation). Moreover Psarropoulos et al. [27]

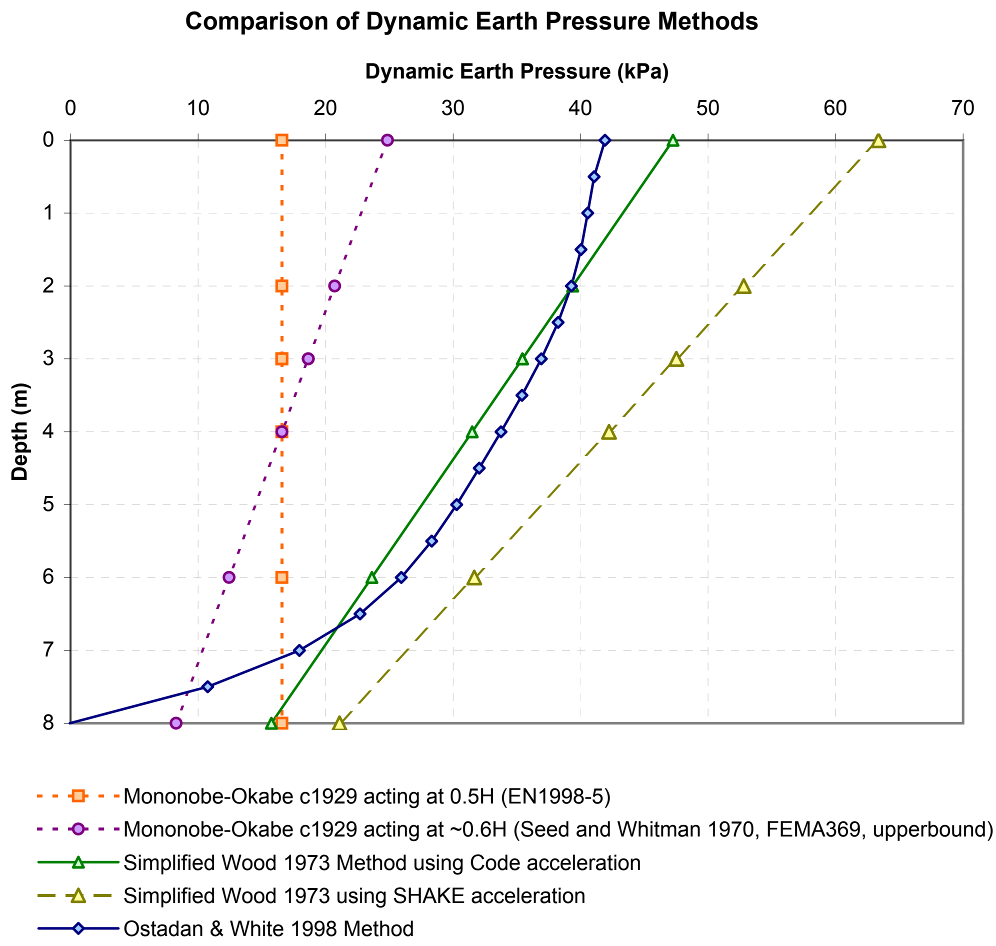


Figure 6. Comparison of calculated dynamic earth pressures for a rigid embedded wall, founded on rock. Sand with ϕ' =35°, Dr =65%. PGA (rock) = 0.15g. PGA (soil) = 0.2g (UBC97), PGA (soil) =0.27g (SHAKE). NB: M-O interpretations for comparison only, not strictly applicable for this wall type.

showed their results compared well to FE modelling. US Army Corps Engineers [28] have adopted this method for the structural design of L-wall stems.

Basement walls and tunnels

For basement walls, Ostadan & White [29] (see also Ostadan [30]) developed a simplified method, based on 1-D site response analysis (such as using SHAKE [31], Oasys SIREN [32], or a similar program) following which a response spectrum analysis is performed to derive the maximum dynamic earth pressure. The final profile is obtained using a semi-empirical normalised earth pressure curve, based on a series of dynamic soil structure interaction (DSSI) analyses. Their results showed that depending on input ground motion, soil and wall properties, the M-O and Wood solutions provided lower and upper bounds respectively. Thus over-predictions by using Wood may be assessed and reduced through this approach that considers the dynamic wall-soil system response to the design ground motion inherently. Figure 6 shows a comparison between methods to derive the dynamic earth pressure profile that was carried out for a typical project, including common interpretations of the use of M-O. This method has been referenced by FEMA 450 [33].

A displacement based design approach to assess racking effects in box tunnels, by Wang [34], is based on a similar approach – parametric DSSI analyses have been performed to derive a method to estimate the displacement and hence strains induced in the tunnel structure. This approach avoids the problem of estimating dynamic earth pressures by either the M-O or Wood methods, which would be inappropriate for most tunnel structures.

An alternative pragmatic design approach is to use a combination of 1-D site response analysis to obtain free field ground displacements, in combination with a pseudostatic soil-structure interaction analysis performed in 2-D finite element analysis software such as Plaxis, Oasys SAFE, Abaqus, FLAC etc., by a means of prescribed displacements (e.g. Free et al. [35]). Further discussion on the seismic analysis of underground structures is provided by

Hashash et al. [36]. Kontoe et al. [37] note that such simplified methods often provide reasonable results despite the inherent simplifications.

Design inertia, phase effects and amplification

Effective acceleration

Sarma and Yang [38] attempted to add a theoretical basis to the common practice of applying a factor of ½ or ⅓ to Peak Ground Acceleration (PGA) by considering the acceleration required to generate 95% of the "energy" in an earthquake record (measured in Arias Intensity), dubbed the A_{95} parameter. From the results of a study of 135 earthquake recordings, a best fit correlation of $A_{95} = 0.675\times\text{PGA}$ was obtained. In contrast, Japanese practice adopts the proposal by Noda et al. [39] where k_h is determined as follows:

$$k_\text{h} = \begin{cases} \frac{1}{3}\,(\text{PGA})^{\frac{1}{3}} & \text{if PGA} \geq 0.2\text{g}, \\ \text{PGA} & \text{if PGA} < 0.2\text{g}. \end{cases} \tag{2}$$

This is based on back-analysed estimates of k_h from quay wall performance during earthquakes, and forms an upper-bound estimate, whilst a mean value is around 0.6×PGA (PIANC [5]). Al Atik and Sitar [40] found that a value of 0.65×PGA provided reasonable estimates for matching the use of M-O earth pressures centrifuge test results of embedded walls. The actual value will depend on the ground motion, wall geometry, soil properties, and whether liquefaction occurred. None of these empirical approaches considers all of these factors systematically.

Steedman and Zeng [17] investigated the assumption of infinite stiffness on dynamic active earth pressures through a pseudo-dynamic analysis method. Their results suggest phase effects on earth pressures are of small significance, but the effects of amplification were significant, and clearly both these effects would be more pronounced with large walls where peak ground velocity (PGV) rather than PGA will control the maximum inertia applied to the wall during the design earthquake.

EC8 is perhaps the first code to recommend that walls larger than 10m be considered in a site response analysis for design. Arguably this should be performed as a 2-D site response analysis that may consider the change in profile of the ground surface provided by the wall, account for the relative stiffness differences between wall and backfill, and the dynamic response of any structures present that may influence the wall behaviour. A 1-D analysis cannot capture these important aspects. Recently Anderson et al. [22] presented the results of a parametric study to consider these effects more systematically using QUAD4M – an equivalent linear 2-D site response program [41]. They produced a chart showing significant reduction in design ground motion from PGA may be taken depending on the height of the wall, and the ground motion characteristics – the latter simplified as a ratio of the design spectra at a period of 1s and at PGA (i.e. relative contribution of long period to short period motion).

Displacement estimation

The most common method used to reduce the design acceleration from PGA, is to utilise the Newmark sliding block concept [42], originally developed for estimating co-seismic displacements of non-catastrophic embankment failures. Richards and Elms [43] adopted this method for co-seismic sliding of retaining structures in an early application of the performance based design concept. EC8 considers this method inherently in the analysis procedure, by allowing a reduction in design inertia k_h used in conjunction with the M-O method, to 0.5×PGA, implicitly allowing for small co-seismic displacements to occur during the design earthquake event (typically less than 10cm and therefore negligible for most applications). A simple means to estimate the order of magnitude of the displacements is also provided. However, the meaning of this reduction and the estimate of displacement is ambiguous for embedded walls, and caution is advised. It is recommended that the mode of deformation be considered, and for embedded walls the method of Veletsos & Younan [18] based on a shear beam model may be more appropriate.

If one wishes to consider a Newmark analysis directly, or using one of the many published empirical methods to assess sliding displacements, the implicit reduction factor in EC8 should be removed, and partial factors set to unity, prior to calculation of the critical acceleration. For tilting mode displacements, the method of Steedman and Zeng [44] (see also [45]) may be applied where the wall is situated on a rigid founding stratum. For combined tilting and bearing mode displacements (most applications) a modified method may be developed based on the same concepts. An alternative to the above simple rigid block models is to use a fully dynamic numerical analysis, the details of which is beyond the scope of this paper, but there are many examples in the literature.

Conclusions

This technical note summarises the common methods to assess dynamic earth pressures and their limitations. It also provides reference to modifications and enhancements to the base design methods in order to account for these limitations. Where possible, reference is made to Eurocode 8 to note to what extent these developments have been incorporated into modern design. Hopefully this paper provides a useful reference for understanding the subtleties of dynamic earth pressure evaluation and updates the reader on recent developments.

References

[1] TAYLOR M.L. Performance based design of retaining structures – part 1: theory. *Proceedings of the 18th New Zealand Geotechnical Society 2008 Symposium*.

[2] TAYLOR M.L., KONTOE S., & SARMA S. Performance based design of retaining structures – part 2: case study of a quay wall. *Proceedings of the 18th New Zealand Geotechnical Society 2008 Symposium.*

[3] BRITISH STANDARDS INSTITUTION. *Eurocode 8: Design of structures for earthquake resistance — Part 5: Foundations, retaining structures & geotechnical aspects.* BSI, 2004, BS EN 1998-5.

[4] STEEDMAN R.S., Seismic design of retaining walls. *Proc. Instn Civ. Engrs Geotech. Engng*, 1998, **131**, 12-22.

[5] PIANC. *Seismic Design Guidelines for Port Structures.* International Navigation Association. Balkema, Rotterdam, 2001.

[6] MONONOBE, N., & MATSUO, H., On the determination of earth pressures during earthquakes, *Proceedings of the World Engineering Conference*, **9**, 176. 1929

[7] OKABE, S. General theory of earth pressure. *Journal of the Japanese Society of Civil Engineers*, Tokyo, **12**(1), 1926

[8] SEED H.B., & WHITMAN R.V. Design of earth retaining structures for dynamic loads. *ASCE Special Conference on Lateral Stresses in the ground & the design of earth retaining structures*, Cornell, New York, ASCE 1970. 103-147

[9] MATSUZAWA, H., ISHIBASHI, I., & KAWAMURA, M., Dynamic soil & water pressures of submerged soils, *Journal of Geotechnical Engineering*, ASCE 1985, **111**, No. 10, pp 1161-1176

[10] KRAMER, S.L., *Geotechnical Earthquake Engineering*. Prentice Hall 1996, USA

[11] KUMAR J. Seismic Passive Earth Pressure Coefficients for Sands. *Canadian Geotechnical Journal*, 2001, **38**, 876-881.

[12] BRITISH STANDARDS INSTITUTION. E*urocode 7: Geotechnical design- Part I general rules.* BSI 2004, BS EN 1997-1

[13] NAVAL FACILITIES ENGINEERING COMMAND. *Foundations & Earth Retaining Structures – Design Manual.* Naval Facilities Engineering Command (NAVFAC), Washington D.C. 1986, DM7.02.

[14] KOSEKI, J., F. TATSUOKA, Y. MUNAF, M. TATEYAMA & K. KOJIMA. A modified procedure to evaluate active earth pressure at high seismic loads. *Special Issue Soils & Foundations*, 1998, **2**, 209-216.

[15] INTERNATIONAL ORGANISATION for STANDARDISATION. *Bases for design of structures — Seismic actions for designing geotechnical works*. ISO 2005, ISO 23469 [draft]

[16] JAPANESE GEOTECHNICAL SOCIETY. *Principles for Foundation Designs Grounded on a Performance-based Design Concept*. March 2006, Geocode 21. JGS 4001-2004

[17] STEEDMAN R.S., & ZENG X. The influence of phase on the calculation of pseudo-static earth pressure on a retaining wall. *Géotechnique*, 1990, **40**, No. 1, 103-112.

[18] VELETSOS, A.S., & YOUNAN, A.H. Dynamic Response of Cantilever Retaining Walls. *Journal of Geotechnical & Geoenvironmental Engineering*. ASCE 1997, **123** (2):161-172

[19] WOOD J.H., *Earthquake Induced Soil Pressures on Structures.* Report No. EERL 73-05, Earthquake Engineering Research Laboratory, Calif. Inst. Tech., Pasadena. 1973.

[20] RICHARDS, R., HUANG, C., & FISHMAN, K.L., (1999) Seismic Earth Pressure on Retaining Structures. *Journal of Geotecnical & Geoenvironmental Engineering*. ASCE 1999, **125** (9), pp. 771-778

[21] WHITMAN, R.V. Seismic design of earth retaining structures. *Proceeding of the 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering & Soil Dynamics*, Missouri, University of Missouri, Rolla, USA. 1991.

[22] ANDERSON D.G., MARTIN G.R., LAM I.P. & WANG J.N. *Seismic Analysis & Design of Retaining Walls, Buried Structures, Slopes, & Embankments*. Vol.1. NCHRP Report 611. Transportation Research Board 2008, Washington D.C.

[23] THE OVERSEAS COASTAL AREA DEVELOPMENT INSTITUTE OF JAPAN, *Technical standards & commentaries for port & harbour facilities in Japan.* Eds: Goda Y., Tabata T., & Yamamoto S. OCDI 2002.

[24] EARTHQUAKE ENGINEERING FIELD INFESTIGATION TEAM. *The Kocaeli, Turkey Earthquake of 17/8/99, EEFIT photo gallery.* [Digital Media]. 2003.

[25] WESTERGAARD, H., Water pressure on dams during earthquakes, *Transactions of the ASCE*, ASCE 1933, **98**, pp 418-433.

[26] MATTHEWSON, M.B., WOOD, J.H. & BERRILL, J.B. Seismic Design of Bridges – Earth Retaining Structures, *Bulletin of the New Zealand Society of Earthquake Engineering*, 1980. **13**(3), 280-293

[27] PSARROPOULOS, P.N., KLONARIS, G., & GAZETAS, G., Seismic earth pressures on rigid & flexible retaining walls. *Soil Dynamics & Earthquake Engineering*. 2005, **25**, 795-809

[28] STROM R.W. & EBELING R.M. *Seismic Structural Considerations for the Stem & Base of Retaining Walls Subjected to Earthquake Ground Motions.* ERDC/ITL TR-05-3. US Army Corps of Engineers, Engineer Research & Development Center. 2005. Washington D.C.

[29] OSTADAN, F. & WHITE, WH. Lateral Seismic Earth Pressure: an Updated Approach, *US-Japan SSI Workshop* 1998, USGS, Menlo Park, CA.

[30] OSTADAN, F. *Seismic Soil Pressures for Building Walls: An Updated Approach*, *Soil Dynamics & Earthquake Engineering*, 2005, **25**, 785-793.

[31] SCHNABEL, P.B., LYSMER, J., & SEED, H. B. *SHAKE: A computer program for earthquake response analysis of horizontally layered sites*. Report EERC 72-12, Earthquake Engineering Research Centre, University of California, Berkeley, CA. 1972.

[32] HEIDEBRECHT, A.C., HENDERSON, P., NAUMOSKI, N. & PAPPIN, J.W. Site response effects for structures located in sand sites. *Canadian Geotechnical Journal*, **27**, 1990, 342-354

[33] BUILDING SEISMIC SAFETY COUNCIL. *NEHRP Recommended Provisions for seismic regulations for new buildings & other structures (FEMA 450). Part 2: Commentary.* National Institute of Building Sciences, Washington D.C. 2004

[34] WANG J.N. *Seismic Design of Tunnels. A Simple State-of-the-Art Design Approach.* Parsons Brinckerhoff Monograph 7. 1993, New York, NY.

[35] FREE M.W., PAPPIN, J.W., SZE J.W.C., & McGOWAN M.J. Seismic design methodology for buried structures. *Proceedings 14th South East Asian Geotechnical Conference*, 2001. pp 509-514.

[36] HASHASH Y.M.A., HOOK J.J., SCHMIDT B., & YAO J. I.C. Seismic design & analysis of underground structures. *Tunnelling & Underground Space Technology*, 2001, **16**, 247-293

[37] KONTOE S., ZDRAVKOVIC L., POTTS, D.M., MENKITI C.O., Case study of seismic tunnel response. *Canadian Geotechnical Journal*, 2008, **45**, 1743-1764.

[38] SARMA, S.K., & YANG K.S. An evaluation of strong motion records, & a new parameter A95. *Earthquake Engineering & Structural Dynamics*, 1987, 15, 119-132.

[39] NODA, S., UWABE, T. & CHIBA, T. Relation between seismic coefficient & ground acceleration for gravity wall. *Report of the Port & Harbour Research Institute,* Japan 1975, **14**, No. 4, 67–111 [in Japanese].

[40] AL ATIK, L. & SITAR, N. *Development of Improved Procedures for Seismic Design of Buried & Partially Buried Structures,* PEER Report 2007/06, Pacific Earthquake Engineering Research Center 2007.

[41] HUDSON M., IDRISS I.M. & BEIKAE M. *User's Manual for QUAD4M: A computer program to evaluate the seismic response of soil structures using finite element procedures & incorporating a compliant base.* University of California at Davis 1994.

[42] NEWMARK N. M. Effect of earthquakes on dams & embankments. 5th Rankine lecture. *Géotechnique*. 1965, **15**, 2, 139-160

[43] RICHARDS, R., & ELMS, D., Seismic Behaviour of Gravity Retaining Walls, *Journal of the Geotechnical Engineering Division*. ASCE 1979, **15**, GT4, 449-464

[44] STEEDMAN, R.S., & ZENG X., Rotation of Large Gravity Walls on Rigid Foundations under Seismic Loading, In: *Analysis & Design of Retaining Structures Against Earthquakes*. Ed: Prakash S., ASCE 1996.

[45] ZENG X. & STEEDMAN, R.S., Rotating Block Method for Seismic Displacement of Gravity Walls, *Journal of Geotechnical & Geoenvironmental Engineering.* ASCE 2000, **26** (8) 709-717.

SECED Newsletter

The SECED Newsletter is published quaterly. Contributions are welcome and manuscripts should be sent on a CD or by email. Diagrams, pictures and text should be in separate electronic files. Copy typed on paper is also acceptable. Diagrams should be sharply defined and prepared in a form suitable for direct reproduction. Photographs should be high quality. Colour images are welcome. Diagrams and photographs are only returned to authors on request.

Contributions should be sent to the Editor of the Newsletter, Andreas Nielsen.

Email
andreas.nielsen@jacobs.com

Post
Editor SECED Newsletter
c/o The Secretary
SECED
Institution of Civil Engineers
One Great George Street
London, SW1P 3AA
United Kingdom

SECED

SECED, The Society for Earthquake and Civil Engineering Dynamics, is the UK national section of the International and European Associations for Earthquake Engineering and is an affiliated society of the Institution of Civil Engineers. It is also sponsored by the Institution of Mechanical Engineers, the Institution of Structural Engineers, and the Geological Society. The Society is also closely associated with the UK Earthquake Engineering Field Investigation Team. The objective of the Society is to promote co-operation in the advancement of knowledge in the fields of earthquake engineering and civil engineering dynamics including blast, impact and other vibration problems. For further information about SECED contact:

The Secretary
SECED
Institution of Civil Engineers
One Great George Street
London, SW1P 3AA, UK

Or visit the SECED website:
http://www.seced.org.uk

New books

Book review

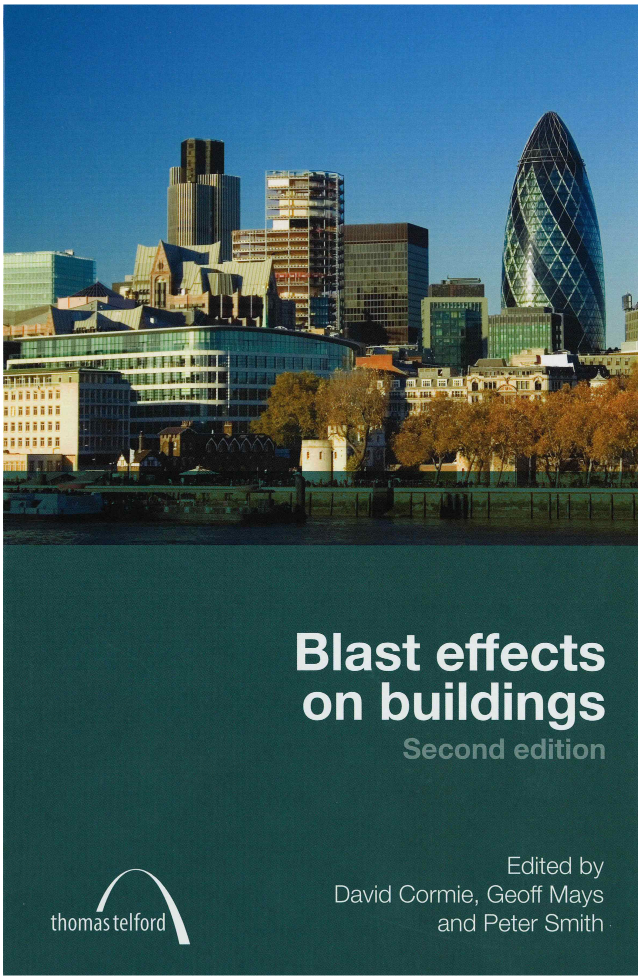
Blast effects on buildings

Edited by David Cormie, Geoff Mays & Peter Smith

Thomas Telford £65.00 pp356

Andrew Mair

Engineering Director, Jacobs



This is the second edition of a book which has become a standard text on the subject of design for blast effects since it was first published in 1995. The first edition of the book concentrated on the mechanics of design for blast loading, including blast wave propagation and structure interaction effects, structural dynamics, and design of concrete structures, steel structures and glazing. The second edition expands on this to take a more holistic view of addressing the threat from explosions, including whole-building response to blast and a discussion of various counter terrorism measures, which are of course becoming more prevalent in today's society.

Generally blast loading on buildings is derived using empirical methods which have their origins in tests conducted by the US military in the 1950's and 60's, and which were presented in the Standard TM5-1300 (now superseded by UFC 3-340-02). Whilst this book still concentrates on empirical techniques for the derivation of blast loading, the authors discuss the limitations of these techniques and briefly introduce phenomenological methods for deflagrative events, and introduce first principle methods using computational fluid dynamics to quantify complex blast-structure interaction effects.

The design of structures in reinforced concrete and steel has been brought up to date with reference to the appropriate Eurocodes, including guidance on the selection of partial load factors and combination factors. Guidance is provided on the use of static increase factors (SIF's) for material strengths to account for differences between characteristic and actual yield strengths, and dynamic increase factors (DIF's) to account for the high strain rates experienced during a blast excursion. Detailing is an important aspect of designing structures to resist dynamic effects, and advice is provided on the selection of material grades and detailing rules. A new chapter is included in the Second Edition on the design of steel-concrete-steel (SCS) composite structures. The failure modes of SCS construction are analogous to reinforced concrete construction, but with some subtle differences which are clearly explained in the text.

As the design of structures in steel, reinforced concrete and SCS composite is set out as three individual chapters, there is invariably some repetition in the text of these chapters. However, the alternative of attempting to address three different materials in one chapter may have led to confusion of the reader, and the repetition does not detract from a useful and informative text with clear examples.

Glazing is becoming an increasingly important element in the design of buildings, with full height glazed facades now being common place in modern buildings. The failure of glazing in a blast event can lead to large scale casualties. The pros and cons of various types of glazing and protection systems are discussed, and a design approach presented which considers the pre-crack resistance of the glass and the post crack membrane capacity.

There is a useful chapter dedicated to whole-building response to blast damage, which looks at the prevention

New books

of disproportionate and progressive collapse. The authors recognised that there is no universal approach to designing for robustness, and differing approaches are taken depending on the codes and standards being applied. It is also recognised that most of the robustness rules which have been developed around the world relate primarily to accidental actions, rather than those of malicious origin, which most blast scenarios reflect. The key issues which an Engineer needs to consider in assessing the whole-building response to blast are set out in a clear concise manner.

The book is well written and easy to digest for those with a basic knowledge of structural dynamics. I would thoroughly recommend this text for those interested in blast effects on buildings, whether they be students, those seeking an introduction to the subject matter or seasoned practitioners.

Book announcement

Seismic Design of Buildings to Eurocode 8

Edited by Ahmed Y. Elghazouli

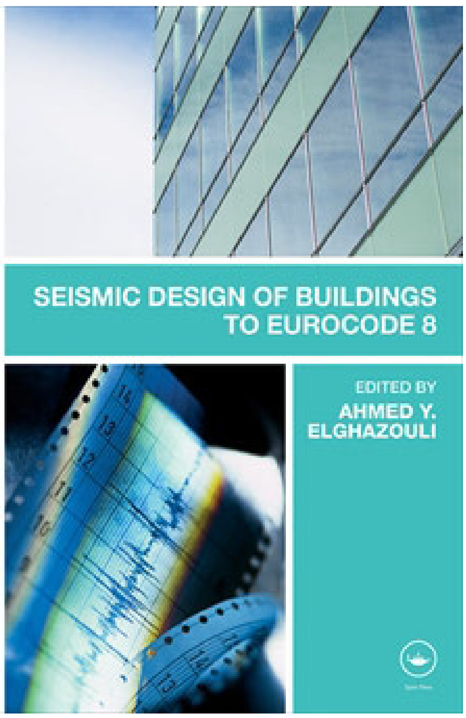
Taylor & Francis £65.00 pp336

Ahmed Elghazouli

Imperial College London

New book on EC8

Eurocode 8 has acquired the status of a full Euronorm and has been implemented in several national codes both within and beyond Europe. As part of addressing the need for practical information and training, a new book has recently been published by Taylor and Francis (ISBN 978-0-415-44762-1, 336 pp). The book links the principles of seismic design to the provisions of Eurocode 8, illustrated with design examples. Concrete and steel buildings, and their foundations, are given special emphasis but the principles are also applicable to other types of structure and sub-structure.



The book consists of nine chapters, starting with an introduction to the contents and implementation of EC8 in Chapter 1 (by Philippe Bisch). The second chapter (by Julian Bommer and Peter Stafford) provides a detailed review of methods used in determining seismic hazards and earthquake actions, with specific reference to the stipulations of EC8. Chapter 3 (by Martin Williams) presents a review of basic dynamics including the response of single and multi-degree-of-freedom systems and the use of earthquake response spectra, leading to the seismic analysis methods used in EC8. This chapter also introduces an example building that is used throughout the book to illustrate the use of EC8 in practical building design. The provisions relating to general considerations for the design of buildings (including regularity, capacity design, and foundation/siting) are discussed in Chapter 4 (by Edmund Booth and Zygmunt Lubkowski). Chapter 5 (by Andy Campbell and Mario Lopes) focuses on reinforced concrete structures and culminates in a detailed design example. Similarly, steel and composite steel/concrete structures, together with design examples, are dealt with in Chapters 6 and 7 (by Ahmed Elghazouli and Miguel Castro). The behaviour and design of both shallow and pile foundations, coupled with illustrative design examples, are covered in Chapter 8 (by Gopal Madabhushi, Indrasenan Thusyanthan, Zygmunt Lubkowski and Alain Pecker) and Chapter 9 (by Gopal Madabhushi and Robert May).

The book stems mainly from practical short courses on seismic design, run jointly by SECED and Imperial College London, with contributions from leading practitioners and senior academics. The next short course is planned for September 2010; details will be announced in the SECED Newsletter and website in due course. For further information, contact Prof. Ahmed Elghazouli at Imperial College London (e-mail: a.elghazouli@imperial.ac.uk).

Notable Earthquakes January – June 2009

Reported by British Geological Survey

Issued by: Davie Galloway, British Geological Survey, August 2009.
Non British Earthquake Data supplied by The United States Geological Survey.

Year	Day	Mon	Time UTC	Lat	Lon	Dep km	Magnitude M_L	M_b	M_W	Location
2009	03	JAN	19:43	0.41S	132.89E	17			7.6	PAPUA, INDONESIA
At least five people killed, over 250 injured and more than 840 buildings damaged in Manokwari and Sorong, western Papua.										
2009	04	JAN	05:10	36.73N	22.28E	10		4.3		SOUTHERN GREECE
One person killed and one injured (by a falling wall) in Khora Gaitson.										
2009	08	JAN	19:21	10.17N	84.20W	14			6.1	COSTA RICA
At least 40 people killed (including 17 still missing presumed dead), over 100 injured and more than 500 houses destroyed or damaged in central Costa Rica. The majority of the casualties were as a result of landslides in the region.										
2009	10	JAN	23:26	56.75N	4.37W	7	2.4			LOCH ERICHT, HIGHLAND
2009	15	JAN	05:32	60.27N	1.28W	29	3.3			SHETLAND ISLANDS
Felt throughout the Shetland Islands (4 EMS).										
2009	15	JAN	07:27	22.35S	170.64E	27			6.7	LOYALTY ISLANDS
2009	15	JAN	17:49	46.86N	155.15E	36			7.4	KURIL ISLANDS
2009	19	JAN	03:35	22.60S	170.91W	12			6.5	LOYALTY ISLANDS
2009	20	JAN	06:51	49.20N	3.23W	13	2.1			ENGLISH CHANNEL
2009	11	FEB	17:34	3.88N	126.40E	22			7.2	TALAUD, INDONESIA
At least 64 people injured and damage to 600 buildings in Kepulauan Talaud.										
2009	18	FEB	21:53	27.42S	176.33W	25			6.9	KERMADEC ISLANDS
2009	19	FEB	10:11	49.45N	0.52W	5	2.5			ENGLISH CHANNEL
2009	19	FEB	11:44	49.45N	0.54W	5	2.1			ENGLISH CHANNEL
2009	20	FEB	03:48	34.20N	73.90E	10			5.4	PAKISTAN
At least 44 people injured and several landslides reported in Kashmir.										
2009	22	FEB	13:18	56.45N	5.29W	5	1.8			BONAWE, ARGYLL/BUTE
2009	23	FEB	00:58	50.60N	1.01E	5	1.6			ENGLISH CHANNEL
2009	28	FEB	10:11	64.72N	0.57E	10	3.0			NORWEGIAN SEA
2009	03	MAR	14:35	51.12N	1.18E	3	3.0			FOLKESTONE, KENT
Felt Folkestone (4 EMS).										
2009	04	MAR	12:04	51.53N	3.05W	13	1.6			NEWPORT, SOUTH WALES
2009	06	MAR	10:50	80.32N	1.85W	9			6.5	NORTH OF SVALBARD
2009	06	MAR	14:43	49.54N	0.83W	5	2.1			ENGLISH CHANNEL
2009	08	MAR	07:37	49.58N	1.23W	5	2.2			ENGLISH CHANNEL
2009	19	MAR	18:17	23.05S	174.66W	33			7.6	TONGA
2009	20	MAR	07:11	49.50N	0.58W	6	2.3			ENGLISH CHANNEL
2009	21	MAR	15:39	49.51N	0.16W	5	2.1			ENGLISH CHANNEL
2009	26	MAR	04:44	22.68N	85.73E	10		4.1		JHARKHAND, INDIA
Five people injured and several buildings damaged in Chaibasa.										
2009	06	APR	01:32	42.33N	13.33E	9			6.3	CENTRAL ITALY
At least 259 people killed, over 1,000 injured and more than 15,000 buildings damaged or destroyed leaving over 55,000 homeless in the L'Aquila area.										

Notable Earthquakes (continued)

			Time			Dep	Magnitude			
Year	Day	Mon	UTC	Lat	Lon	km	M_L	M_b	M_W	Location
2009	07	APR	04:23	46.05N	151.55E	31			6.9	KURIL ISLANDS
2009	07	APR	17:47	42.28N	13.46E	15			5.5	CENTRAL ITALY
One person killed and additional damage to buildings in the L'Aquila area.										
2009	09	APR	01:46	27.14N	70.75E	44		5.2		INDIA/PAKISTAN BORDER
Six people injured and several buildings damaged in Jaisalmer, India.										
2009	11	APR	11:39	53.69N	0.25W	15	3.0			GOXHILL, NORTH LINCS
Felt Hull, Swanland, Ulceby, Beverley and Binbrook (3 EMS).										
2009	11	APR	23:33	50.19N	5.15W	3	1.4			STITHIANS, CORNWALL
Felt Crowlas and Rinsey, Cornwall (3EMS).										
2009	16	APR	14:57	60.20S	26.86W	20			6.7	STH SANDWICH ISLANDS
2009	16	APR	18:48	49.55N	0.87W	5	2.3			ENGLISH CHANNEL
2009	16	APR	21:27	34.19N	70.08E	6		5.4		AFGHANISTAN
At least 19 people killed, 51 others injured and more than 200 homes destroyed in Nangarhar Province.										
2009	16	APR	23:42	34.11N	70.06E	4		5.1		AFGHANISTAN
Further damage reported in Nangarhar Province.										
2009	18	APR	19:17	46.02N	151.43E	35			6.6	KURIL ISLANDS
2009	20	APR	14:07	49.59N	0.96W	5	2.2			ENGLISH CHANNEL
2009	21	APR	10:05	50.09N	0.35E	7	2.2			ENGLISH CHANNEL
2009	22	APR	10:00	50.08N	0.82E	6	2.3			ENGLISH CHANNEL
2009	22	APR	14:12	55.83N	3.19W	5	2.0			PENICUIK, MIDLOTHIAN
2009	22	APR	16:02	50.10N	0.84E	5	1.9			ENGLISH CHANNEL
2009	23	APR	17:41	50.01N	0.95E	7	2.1			ENGLISH CHANNEL
2009	25	APR	19:01	50.02N	0.92E	5	2.2			ENGLISH CHANNEL
2009	26	APR	05:04	50.01N	0.73E	5	2.2			ENGLISH CHANNEL
2009	27	APR	14:02	50.04N	0.91E	5	2.3			ENGLISH CHANNEL
2009	28	APR	08:17	50.08N	0.68E	6	2.2			ENGLISH CHANNEL
2009	28	APR	10:22	54.17N	3.02W	9	3.7			ULVERSTON, CUMBRIA
Felt widely across Cumbria and Lancashire (5 EMS).										
2009	28	APR	14:50	50.06N	0.78E	6	2.4			ENGLISH CHANNEL
2009	29	APR	09:14	50.06N	0.72E	8	1.9			ENGLISH CHANNEL
2009	29	APR	14:02	50.06N	0.69E	6	2.4			ENGLISH CHANNEL
2009	29	APR	15:01	50.14N	0.90E	5	2.2			ENGLISH CHANNEL
2009	07	MAY	11:08	56.40N	5.69W	13	1.8			ISLE OF MULL
2009	16	MAY	00:53	31.52S	178.79W	55			6.5	KERMADEC ISLANDS
2009	19	MAY	17:35	25.29N	37.74E	2			5.7	WESTERN SAUDI ARABIA
Seven people injured and several large ground cracks observed in Al Madinah.										
2009	28	MAY	08:24	16.72N	86.23W	10			7.3	OFFSHORE HONDURAS
Seven people killed, 40 others injured and more than 130 buildings either damaged or destroyed in northern Honduras.										
2009	29	MAY	06:20	17.03S	168.33E	13			5.7	VANUATU
Ten people injured and several buildings damaged on Tongoa Island.										

Notable Earthquakes (continued)

Year	Day	Mon	Time UTC	Lat	Lon	Dep km	Magnitude M_L	M_b	M_W	Location
2009	02	JUN	02:17	17.76N	167.95E	15			6.3	VANUATU
Four people injured and several buildings damaged on Tongoa Island.										
2009	04	JUN	16:45	49.54N	0.19W	10	1.7			ENGLISH CHANNEL
2009	04	JUN	17:44	49.50N	0.18W	10	1.8			ENGLISH CHANNEL
2009	05	JUN	19:24	51.62N	3.65W	6	2.9			MAESTEG, BRIDGEND
Felt Bridgend and Vale of Glamorgan, South Wales and also in Bridgwater, Somerset (3 EMS).										
2009	10	JUN	04:48	49.58N	0.20W	10	2.3			ENGLISH CHANNEL
2009	10	JUN	04:59	51.81N	3.42W	9	2.0			MERTHYR TYDFIL
2009	13	JUN	17:17	44.72N	78.86E	15			5.4	EASTERN KAZAKHSTAN
One person killed and several buildings damaged in Tekeli.										
2009	14	JUN	09:07	56.10N	4.33W	3	1.6			BUCHLYVIE, STIRLING
2009	23	JUN	14:19	5.16S	153.78E	64			6.7	PAPUA NEW GUINEA
2009	28	JUN	00:05	56.24N	3.73W	5	1.6			BLACKFORD, TAYSIDE

Forthcoming events

Date	Venue	Title	Organiser
27/10/2009	IStructE London	*Engineering Analysis of an Offshore Pipeline on an Escarpment in a Seismically Active Zone: A Case Study in the Caspian* Speaker: Dr Tapan K Sen	IStructE
28/10/2009 at 6pm	ICE London	*Pile Design in Seismic Areas* Speaker: Dr Subhamoy Bhattacharya	SECED Zygmunt Lubkowski (Arup)
17/11/2009 at 6.15pm	ICE London	Scruton Lecture: *Wind-induced Vibrations of Structures* Speaker: Svend Ole Hansen	ICE The Wind Engineering Society
25/11/2009 at 6pm	ICE London	*Japanese Nuclear Power Plant and the 2007 Niigata Earthquake* Speaker: Dr Willy Aspinall	SECED David Mallard (David Mallard & Associates)
01/12/2009	7 Warwick Court London	One day course: *Floor vibrations in steel framed structures*	IStructE SCI
27/01/2010 at 6pm	ICE London	*The State of the Practice of Seismic Hazard Analysis: From the Good to the Bad* Speaker: Dr Norm Abrahamson	SECED Julian Bommer (Imperial College London)
24/02/2010 at 6pm	ICE London	*Soil-Structure Interaction Using Finite Element Methods* Speaker: Dr Harvinder Sehmi	SECED Ian Smith (Atkins)
17/03/2010	Imperial College London	*Half-day seminar on dynamic soil properties* (followed by the 2010 Rankine Lecture by Prof C R I Clayton)	SECED, BGA Andrew Coatsworth (NII) Stavroula Kontoe (Imperial College London)

1st Announcement of the

SECED Young Engineers' Conference

to be held at University College London in November 2010†

The purpose of the 2010 SECED Young Engineers' Conference is to bring together young engineers from both industry and universities to meet and interchange ideas related to the study and practice of earthquake engineering and civil engineering dynamics. The conference will focus on:

- Structural dynamics (experimental, numerical, practical)
- Geotechnical earthquake engineering (experimental, numerical, practical)
- Seismology and risk assessment
- Seismic retrofit
- Seismic resilience

A number of world leading academics and practitioners will be presenting key-note lectures. There will be both poster and oral presentations. Two prizes of £100 will be given to the two best papers presented at the conference, which will also be printed in the SECED Newsletter. (To qualify for prizes, the lead author and presenter must be under 35 years old by 1st November 2010.)

Who should attend

- Practicing engineers under the age of 35 years
- Young academics under the age of 35 years
- Post-doctoral researchers
- Research students
- MSc students

Why you should attend

- To participate in an active exchange of scientific and technical information of use both to practice and academia
- To hear how engineers solve practical earthquake and dynamics problems
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Important dates

Submission of abstracts:	30 April 2010‡
Paper/poster acceptance notification:	30 May 2010
Paper submission:	30 August 2010

† Final date to be announced.

‡ Abstracts must contain an introduction, method and conclusion section and fit into 1 page of A4.

For further information regarding SECED events please contact Pauline Arundel, Engineering Department, at the ICE, on telephone 020 7665 2236, email Pauline.Arundel@ice.org.uk, or visit the SECED website at http://www.seced.org.uk.

Letters

In response to Stewart Gallocher's comments in the last Newsletter Edmund Booth and Bryan Skipp have sent the following letter.

Dear Sir

We have read Stewart Gallocher's comments in the June 2009 edition of the Newsletter, and have the following observations.

Firstly, we greatly welcome the interest shown. The proposals for seismic design procedure given in the UK National Annex (NA) to Eurocode 8, and the associated commentary in the BSI 'published document' PD6698 were finalised after a very wide debate and consultation exercise, but are recognised to be a change from current practice. It was anticipated that these proposals would need to be reviewed after some years of use, to see if they required revision. Therefore, a public debate is very valuable to assist this process.

Secondly, Stewart rightly points out that the proposed procedure is different to that currently adopted in the US. However, it appears that he may not have fully appreciated exactly what is proposed. As the Americans realized some time ago, and subsequently implemented in their codes, designing to a return period of 475 years gives a very non-uniform level of seismic reliability (i.e. lifetime probability of collapse) between regions of high and low seismicity. The problem is even more extreme in an area of very low seismicity, such as the UK, and this is reflected in the very long return period of 2,500 years, used in conjunction with the BGS hazard maps and recommended EC8 spectral shapes, which is given as a conservative option in the UK NA. However, this applies only to the UK and might not be appropriate (nor is this anywhere suggested) for moderately seismic areas of Europe such as parts of southern France, or highly seismic areas such as Greece. Moreover, the US design parameter of

(1,500 year return motion/1.5)

cannot be compared directly with the design parameter in the UK NA of

(2,500 year return motion/1.0),

because the former applies to 'ordinary' structures, while the UK provision applies to consequence class CC3 buildings, which have a high consequence for loss of human life or where the economic or environmental consequences are very great. It may be noted that such buildings in the US attract an `importance factor` greater than 1, whereas the UK NA sets this factor at unity. In fact, the UK NA provides that only the more critical CC3 structures need consider any explicit seismic design provisions, and many are exempt; 'ordinary' (CC2) structures are always exempt. Moreover, the UK NA and PD6698 give the alternative option of selecting values of design ground motion by carrying out a site-specific hazard analysis; the associated return period should be chosen "accounting for the function and consequences of failure of the facility involved". The UK NA is therefore not as conservative in relation to US practice as Stewart appears to believe. Further advice and background information is given in PD6698.

It is also heartening to note that a current major European research project, called SHARE, has as one of its objectives an examination of the appropriate measures for specifying ground motion hazard, including choice of return period. A link has been established between the European committee responsible for Eurocode 8 and SHARE, so the results and recommendations of SHARE will be taken into account in the next major revision to Eurocode 8. It may be of interest to know that the seeds of SHARE – the idea of a harmonised European seismic map without 'step functions' at national borders – were planted by a proposal made 30 years ago by the second author of this letter, Bryan Skipp with Nick Ambraseys. It is good to know these seeds are now bearing fruit.

Yours faithfully

Edmund Booth
UK National Technical Contact for EC8
&
Bryan Skipp
Chair, BSI committee B525/8

Reference

PD 6698:2008, Background paper to the UK National Annexes to BS EN 1998 1, BS EN 1998 2, BS EN 1998-4, BS EN 1998-5 and BS EN 1998-6