TECHNICAL PAPER

JOURNAL OF THE SOUTH AFRICAN INSTITUTION OF CIVIL ENGINEERING

Vol 52 No 1, 2010, Pages 20–27, Paper 688

Background to Draft SANS 10160 (2009): Part 4 Seismic Loading

J A Wium

JAN WIUM, PrEng, is professor in the Murray & Roberts chair for Construction Engineering and Management in the Department of Civil Engineering at Stellenbosch University. He completed his undergraduate and MSc (Eng) degrees at the University of Pretoria and obtained his PhD from the Swiss Federal Institute of Technology in Lausanne. He worked as a consultant for 20 years before joining the University of Stellenbosch in 2003. After first addressing the behaviour of concrete structures and seismic analysis of structures, he now focuses his research on the management and initiation of multidisciplinary capital projects.

Contact details
Department of Civil Engineering
University of Stellenbosch
Private Bag X1
Matieland, 7602
T: 021 808 4498
F: 021 808 4947
E: janw@sun.ac.za

This paper provides a critical overview of the background to the revisions which led to the formulation of Part 4, Seismic Loads, of the South African Standard SANS 10160. The paper also presents a comparison to demonstrate the calibration of the standard against other international standards. Eurocode 8 was used as the principle reference for the formulation of the revised clauses. The clauses for seismic design of SANS 10160 are based on the concept of regular buildings with suitable detailing to allow the necessary ductile behaviour. The paper points out a need to review the use of a redundancy factor which makes provision for a possible lower design peak ground acceleration value and the behaviour factor for reinforced concrete shear walls. The damage limitation criteria also need to be based on Eurocode 8. It is shown that the revised formulation provides lower design forces for regular buildings than in the current standard. This is, however, dependent on the choice of a design nominal peak ground acceleration for the various regions in the country. The available information on nominal peak ground acceleration is presented, and it is pointed out that a decision is needed on the choice of this important parameter for the seismic zones of South Africa.

INTRODUCTION

The revision of the South African Loading Code SABS 0160 (1989) includes a part on seismic actions and requirements for buildings. This paper describes the procedures followed during the revision of the part on seismic loading of SABS 0160 (1989), the motivation for the revision, and the background to specific clauses in the revised sections of SANS 10160 Part 4 (2009). At the time of writing this standard is still at the draft stage. Where necessary a critical evaluation is presented of the chosen formulations. Finally, the paper presents a comparison between results using SANS 10160 (2009) and other standards.

BACKGROUND TO THE REVISION OF SEISMIC LOADING OF SABS 0160

SABS 0160 was first published in 1989, with some amendments in 1993. For the first time, this standard identified regions in the country where building structures had to be designed for seismic loading.

Ever since the publication of SABS 0160 (1989), designers rightly or wrongly considered the seismic provisions of the standard to be unrealistic and too stringent. A meeting held in 2003 with a group of approximately 20 designers in the Western Cape region revealed that some, although aware of SABS 0160 (1989) requirements, often applied the rules to suit economic pressures and the requirements of clients, rather than fully comply with the provisions of the design standard. Some designers preferred to apply international standards rather than use SABS 0160 (1989), although they used nominal peak ground acceleration values defined in SABS 0160 (1989). Some designers also modified the load combinations as presented in SABS 0160 (1989).

A clear need was thus established in 2003 to re-evaluate the seismic provisions of SABS 0160 (1989). Three general objectives were identified:

- The first objective was to determine whether SABS 0160 (1989) was realistic and recent.
- The second objective was to gain the support of the industry by confirming the provisions of the standard, or by issuing a revision to SABS 0160 (1989).
- The third objective was to improve the knowledge in the industry of the basic principles of seismic design of structures.

GENERAL PROCESS AND THE APPROACH ADOPTED

Having recognised the historical objections and lack of confidence among Western Cape designers in SABS 0160 (1989), it was decided in 2004 to establish a regional seismic load working group in the Western Cape, reporting to the working group for the revision of the loading code. Two representatives from Gauteng participated as corresponding members of the regional working group.

Keywords: Structural design standards, South African practice, seismic loads, actions on structures, Eurocode

***Table 1** Recent significant seismic events in South Africa (Historical earthquakes 2007)*

Location	Date	Magnitude (Richter)	Damage
Ceres	1969	6,3	12 killed
St Lucia	1932	6,0–6,5	Serious damage
Welkom	1976	5,2	Building collapsed
Stilfontein	2005	≈5,0	Damage
Mozambique	2006	7,5	4 dead, 36 injured +288 houses destroyed

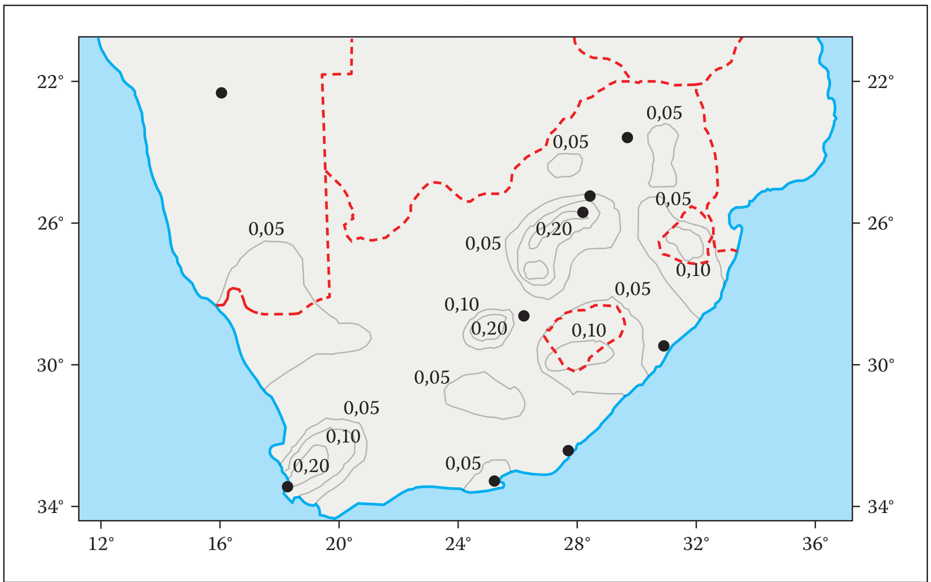

Figure 1 *Seismic hazard map from SABS 0160 (1989) showing peak ground acceleration in g (gravity acceleration) with 10% probability of exceedance in 50 years (SABS 0160 1989)*

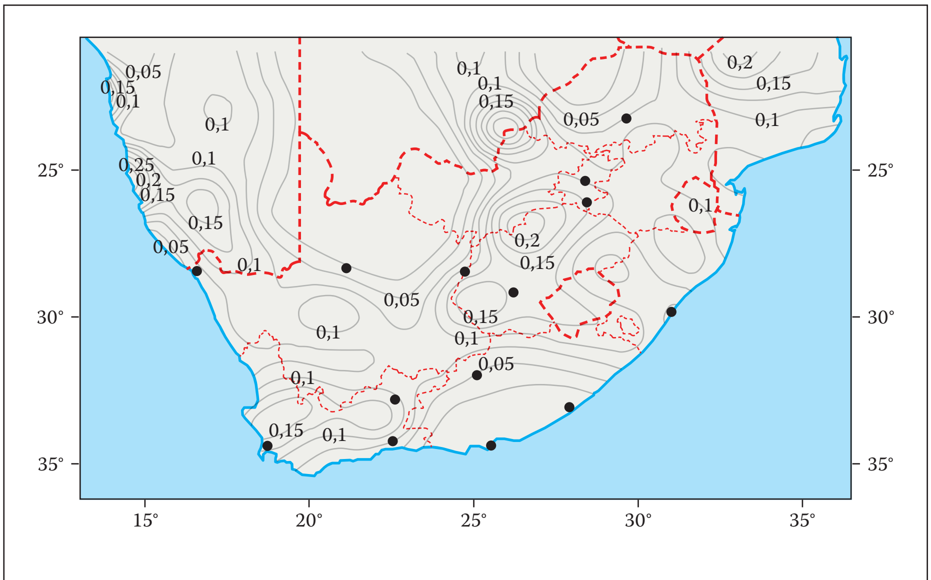

***Figure 2** Seismic hazard map from Council of Geoscience (2003) data, showing peak ground acceleration in g (gravity acceleration) with 10% probability of exceedance in 50 years (for information only; contours are slightly out of position)*

The choice of a regional working group was motivated by the objections by designers in the Western Cape to the provisions for Zone 1 areas in SABS 0160 (1989). The chosen process ensured that designers were involved in the process themselves, thereby creating legitimate support for revisions to SABS 0160 (1989).

It was decided to use SABS 0160 (1989) as a basis and to revise the standard where sufficient information merited any changes. On issues for which sufficient justification did not exist, or which required additional research input, it was broadly decided that the fall-back option would be to keep to the provisions of SABS 0160 (1989). The objective would be to identify those issues that needed more clarification, eventually to be addressed in longer-term research programmes.

Eurocode (2001, 2002) was used as a reference document for the revision of parts of SABS 0160 (1989) other than seismic loading (Retief & Dunaiski 2008). Considering the familiarity of some of the members of the regional working group with a variety of other seismic standards (UBC 1997; NZS 1992; SIA 261 2003; ACI 2002), other international standards were also consulted. The format and sequence of headings as given in the seismic Eurocode 8 (2004) was followed to be consistent with the remainder of the standard. This paper shows that the revised clauses in the seismic loads part of SANS 10160 (2009) are in principle based on Eurocode 8 (2004), apart from behaviour factors for reinforced concrete shear walls and a damage limitation criterion. In the descriptions that follow, reference is often made to the UBC (1997) because of the familiarity of some South African engineers with this standard.

The final process included a calibration of SANS 10160 (2009) against other standards and a comparison with SABS 0160 (1989). Results of this comparison are presented in this paper.

SOUTH AFRICA AND SEISMIC DESIGN OF STRUCTURES

The southern African region is known for its relative seismic stability. Only a small number of medium-intensity earthquakes have occurred since the 17th century. Table 1 summarises the most recent and significant events in southern Africa. On the other hand, between 40 and 60 tremors occur monthly, which occur primarily in the gold mining areas of Gauteng, North West and the Free State. Although the effects of these events are much less serious than those caused by larger earthquakes, extensive damage has occurred in one or two cases (Milford & Wium 1991). For a more comprehensive discussion on the seismicity in the southern African region, refer to Kijko et al (2003) and Milford & Wium (1991).

SABS 0160 (1989) provides a seismic hazard map developed in 1987 and 1989 with contours of peak ground acceleration and a probability of exceedance of 10% in 50 years, which is shown in Figure 1. Internationally this recurrence period is used as a basis for the seismic design of structures (Eurocode 8 2004; SIA 261 2003; UBC 1997). An importance factor is used by standards to change the design value for structures of certain categories to reflect a different recurrence period. A more recent update of the map was published in 2003 by the South African Council for Geoscience (Kijko et al 2003) (refer to Figure 2). Note that the presentation, which is in contour format, is not presented on a flat earth surface and the contours may therefore be slightly out of position. The seismic-event catalogue used to compile this 2003 map was compiled from many different sources, covering a period of time from 1620 to December 2000. The catalogue consists of data on natural as well as mining-induced seismicity.

The seismically active areas in South Africa are broadly divided into two groups in SABS 0160 (1989), namely those where seismic activity is due to natural seismic events (Zone 1 areas), and those where it is predominantly due to mining activity (Zone 2 areas). It has been shown that mine tremors are not likely to produce any significant structural response in buildings with natural vibration frequencies of less than 2 Hz. Stiff structures such as low-rise, load-bearing masonry structures are therefore influenced the most by mining tremors (Milford & Wium 1991).

A comparison between the two seismic hazard maps (1989 and 2003) shows two aspects worth noting. The maximum peak ground acceleration is 0,15 g in the Zone 1 areas (2003) as opposed to 0,2 g on the 1989 map. There is, however, a significantly larger area in the 2003 map in the interior of the country with a nominal peak ground acceleration of at least 0,1 g. Information from the Council for Geoscience does not distinguish between the sources of activity.

Table 2 *Soil classifications to be used in SANS 10160 (2009) with design response spectra*

Soil type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30 cm)	c_u (kPa)
1	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface	> 800	–	–
2	Deposits of very dense sand, gravel or very stiff clay, at least several tens of m in thickness, characterised by a gradual increase of mechanical properties with depth	360–800	> 50	> 250
3	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m	180–360	15–50	70–250
4	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil	< 180	< 15	< 70

Notes:
$v_{s,30}$ average value of propagation velocity of S waves in the upper 30 m of the soil profile at shear stress of 10^{-5} or less
N_{SPT} Standard Penetration Test blow count
c_u undrained shear strength of soil

Regions of natural seismicity

SABS 0160 (1989) specifies a nominal peak ground acceleration (PGA) of 0,1 g for the design of building structures in areas subjected to natural seismic events. No direct reference is available for the choice of the nominal peak ground acceleration of 0,1 g in SABS 0160 (1989). The maximum value shown in Figure 1 is 0,2 g in the Western Cape. A peak ground acceleration (PGA) value of 0,1 g is, however, tabulated in SABS 0160 (1989) for Cape Town. Considering that the higher PGA values are shown to occur slightly to the north of Cape Town, it would appear that the choice of a PGA of 0,1 g in SABS 0160 (1989) is based on the value for Cape Town with its higher density of infrastructure.

The definition of a representative peak ground acceleration value for a structural design standard requires that a variety of parameters need to be considered for a specific region. If one considers the peak ground accelerations (PGA) shown in Figure 2, it is clear that a choice of the design value depends on whether the peak value, an average value, or the "PGA contour" value is used. A decision must also take into consideration the size of area between maximum and minimum values. Furthermore, in the case of the Western Cape, it may for example be reasoned, based on economic grounds, that Cape Town, with its density of infrastructure, should not be subjected to the higher design PGA values expected to the north of the city.

Items to be considered when defining a design peak ground acceleration therefore include:

- A thorough understanding of the risks involved to the infrastructure and to the inhabitants in the region.
- A consensus by stakeholders (scientists, engineers, politicians, communities) on the level of risk considered to be acceptable.
- Knowledge of the impact of a decision on the local economic environment, and an acceptance of these effects.

At the time of the drafting of the design standard, the working group was not in a position to make a final decision on a value to be used as design peak ground acceleration for any region in the country. They emphasised, however, the need to quantify parameters as set out above.

It was therefore proposed that the current value of 0,1 g for the design peak ground acceleration, as defined in SABS 0160 (1989), be adopted as an interim value for Zone 1 areas in the revision of the standard.

Regions of mining-induced seismicity

SABS 0160 (1989) specifies that structures in mining regions comply with certain conceptual layout requirements. The magnitude of peak ground acceleration values in mining regions do not differ from the 1989 information (refer to Figures 2 and 3), and the requirements of SABS 0160 (1989) for structural design in mining areas are therefore incorporated in SANS 10160 (2009) without change.

SEISMIC DESIGN AND A PHILOSOPHY FOR SANS 10160

Conceptual design and correct structural detailing

A report on the Erzincan earthquake in Turkey in 1999 (Earthquake Hazard Centre Newsletter 1999) states that sophistication of calculations and designing for a greater total base shear force do not necessarily lead to improved earthquake resistance of structures. These concepts are also taught by international institutions where earthquake engineering forms part of the structural engineering curriculum (Dazio 2007). Considering the possible magnitude of seismic events in South Africa, it is important to inculcate in designers, developers and owners an awareness that correct structural concepts and appropriate detailing of structural elements will be more effective to resist seismic actions than extensive calculations based on a flawed concept.

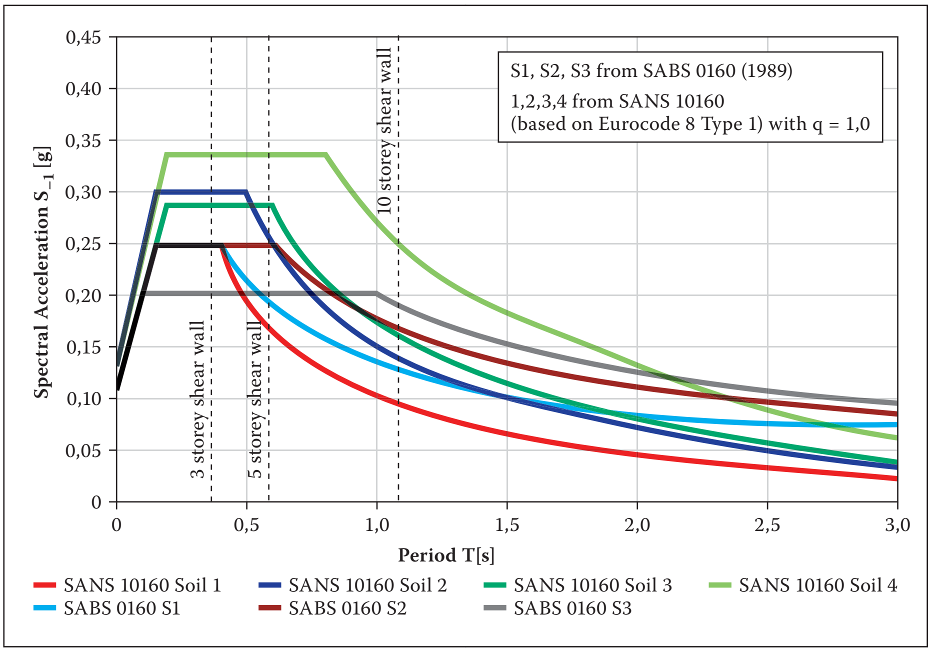

***Figure 3** Comparison between response spectra from SANS 10160 (2009) (taken from Eurocode 2004) and response spectra from SABS 0160 (1989)*

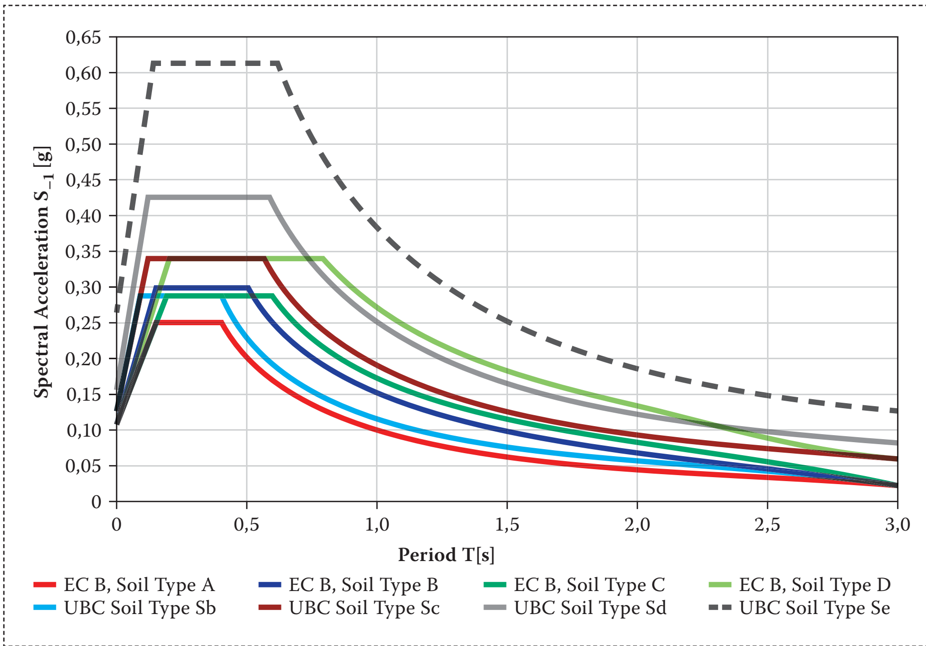

***Figure 4** Comparison between response spectra Type 1 from Eurocode 8 (2004) and the UBC (1997)*

SANS 10160 (2009) therefore aims to provide the basic principles of conceptual layout for acceptable behaviour under seismic loads. The principles presented in SANS 10160 (2009) have to a large extent been taken from SABS 0160 (1989), supplemented with a few additional guidelines from Eurocode 8 (2004). Emphasis is placed on structural simplicity, uniformity of buildings in plan layout and elevation, and on structural redundancy. The possible negative effect of non-structural infill panels (such as stiff masonry panels) on the behaviour of structures is also emphasised in SANS 10160 (2009).

Equally important for a correct structural concept is the appropriate structural detailing of a structure to enable ductile behaviour. SANS 10160 (2009) therefore incorporates rules for structural detailing of reinforced concrete members.

REVISIONS TO SABS 0160 (1989)

The most relevant revisions incorporated into SANS 10160 (2009) are presented in the following paragraphs. The implication of these changes on the design force is presented towards the end of the paper where it is compared to other standards as well as to SABS 0160 (1989).

The following items are presented :

- Soil types and acceleration response spectra
- Design load and load factors
- Redundancy factor
- Behaviour factors
- Design methods
- Calculation of the first natural dynamic period

Response spectra and soil types

The response spectra in SABS 0160 (1989) were obtained from the ATC (1978) which gives response spectra for three soil types. Response spectra in international standards have since been significantly revised (Eurocode 8 2004) (UBC 1997) (SIA 261 2003) and are now also presented for more soil types. A description of soil types taken from Eurocode 8 (2004) and incorporated in SANS 10160 (2009) is presented in Table 2.

A comparison of the design response spectra for different soil types is shown in Figure 3 which gives the spectra from Eurocode 8 (2004) (soil – Type 1) and SABS 0160 (1989). The first natural period of three reinforced concrete shear wall structures are also presented in Figure 3. It can be seen that the spectra in SABS 0160 (1989) differ greatly from those in Eurocode 8 (2004), especially for softer soils and longer-period structures.

Spectra with similar shapes and approximate values to those in Eurocode 8 (2004) are found in the UBC (1997). A comparison between the spectra from Eurocode 8 (2004) and the UBC is presented in Figure 4. Until better information becomes available about spectra for South Africa, it was decided to adopt the design spectra from Eurocode 8 (2004). It should be noted that only *design spectra* are presented in SANS 10160 (2009) and no *elastic response spectra* as given in Eurocode 8 (2004).

Eurocode 8 (2004) provides two series of design spectra which distinguish between Type 1 and Type 2 soil spectra. If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude of not greater than 5,5 it is recommended that the Type 2 spectrum be adopted.

Considering very high peaks in the response spectra at low periods and the rapid reduction of values at higher periods for Type 2 spectra, it was considered more appropriate (and conservative (refer to Table 3)) to choose response spectra Type 1 for SANS 10160 (2009). The Type 2 spectra from Eurocode 8 are not much different from the shape of spectra presented by Milford and Wium (1991) for mining-induced activity. It is suggested that these spectra be evaluated and the development of spectra for mining regions in a future activity be considered.

Load factors

SABS 0160 (1989) requires buildings subjected to seismic loading to be designed for the ultimate limit state with a partial load factor of 1,6 on the seismic load effect, including a load factor of 1,2 for dead load effects. Eurocode 8 (2004), SIA 260 (2004) and NZS (1992) require verification of buildings in the serviceability and the ultimate limit states. For the ultimate limit state a seismic event is considered in a manner similar to accidental loading with a load factor of 1,0 on the seismic load effect. To be consistent with Eurocode 8 (2004), SANS 10160 (2009) now also uses a load factor of 1,0 in the ultimate limit state for the seismic condition.

The design value of the seismic action (A_d) used in the seismic load combination is given by SANS 10160 (2009) as follows:

$$A_d = \gamma\ [\rho \gamma_I (E_x \text{ “+” } 0{,}3.\ E_y) \text{ “+” } E_z] \qquad (1)$$

where γ is the load factor, 1,0; ρ is the redundancy factor described below; γ_I is the importance factor; and E_x, E_y and E_z are the perpendicular components of the seismic action (horizontal x and y, vertical z).

A damage limitation criterion has now been included in SANS 10160 (2009) with a formulation taken from the UBC (1997). This enables designers to determine for themselves the necessary precautions required to prevent damage to non-structural partitions and possible pounding between structural parts across isolation joints. This choice of damage criteria is not consistent with the principle of using Eurocode 8 (2004) as the reference document and needs to be reconsidered for the standard.

Redundancy factor

A redundancy factor (ρ) is included in the calculation of the seismic action (refer to Equation (1)). The factor is not used in Eurocode 8 (2004). It is a function of the number of elements resisting storey shear, and is obtained from the UBC (1997). The factor is considered as a compensation for the points described below. It is presented in SANS 10160 (2009) with the following limitations:

$$1{,}2 < \rho < 1{,}5 \qquad (2)$$

The UBC uses a smaller minimum value of 1,0. The reason for the minimum value of 1,2 in SANS 10160 (2009) is based on the following motivation:

The Loading Code subcommittee is reluctant to accept the current value for a nominal peak ground acceleration of 0,1 g in the light of higher values shown for some of the Zone 1 areas, notably > 0,15 g in the Western Cape (refer to Figure 2). In view of this it is reasoned as follows:

- The redundancy parameter provides a factor of 1,5 on the choice of nominal peak ground acceleration, with the benefit of a reduced value to be obtained if longer shear walls are used.
- The factor compensates for the higher behaviour factors for shear walls as taken from the UBC (1997) in comparison with those in Eurocode 8 (2004).

The choice by a designer of longer walls to obtain a reduced redundancy factor may be counter-productive in some cases, resulting in increased foundation size due to an increased stiffness and high moment over-strength values. These effects need to be taken into account when the designer considers using a reduced redundancy factor.

The incorporation of the redundancy factor in the standard needs to be reconsidered. It would also be more transparent to choose appropriate values for the design peak ground acceleration and the behaviour factors rather than to disguise it behind this redundancy factor. The danger exists that when designers look up the design PGA value, the redundancy factor may be overlooked where it appears in Equation (1).

Behaviour factors

In order to accommodate the magnitude of accelerations and resulting displacements during a seismic event, current practice assumes structures to behave non-linearly in well-defined plastic design zones to absorb the energy from the seismic event. However, calculation methods commonly available to designers are based on linear analysis techniques.

The *behaviour factor* is defined in design standards to represent the ratio between the ductile non-elastic deformation capacity and the linear elastic deformation capacity of a member (Chopra 2002). Behaviour factors also incorporate an allowance for over-strength of the member, which can be a function of the real material characteristics as opposed to design values, and over-design resulting from the choice of actual quantities.

The non-linear behaviour of a structural system is obtained for reinforced concrete structures by confinement of the concrete compression zones and by ductility of reinforcement. The correct reinforcement detailing provides the necessary confinement to increase the strength and ductility of concrete elements. Correct detailing also prevents longitudinal reinforcement elements from buckling under high compressive forces after spalling of cover concrete (Paulay & Priestley 1992). Although reference is made in SABS 0160 (1989) to a need for “sufficient stirrups” in beam and column elements to achieve ductility, no guidance is provided to the designer on what is considered to be “sufficient”.

Booth et al (1998) present a comparison between the seismic design procedures of different design standards. It is pointed out that the basis for selecting ground motions varies between standards, hence the elastic forces to be reduced are not the same, and a direct comparison between behaviour factors from different standards should therefore be done with caution.

In SABS 0160 (1989) the behaviour factor for reinforced concrete construction of *moment frame systems* (2,0) is generally lower than the behaviour factors in Eurocode 8 (2004) (3,3 to 3,9) and the UBC (1997) (3,5), but higher than the value in Eurocode 8 (1,5) when no special detailing rules are applied. For reinforced concrete *shear walls* SABS 0160 (1989) uses a behaviour factor of 5,0 which is less than the value in the UBC (5,5) and greater than the value in Eurocode 8 (3 to 3,3). No special detailing rules are specified in SABS 0160 (1989). From comparison with other standards it is evident that special detailing rules are required to justify the magnitude of the behaviour factors. This omission in SABS 0160 (1989) is now addressed in SANS 10160 (2009) by the inclusion of rules for the detailing of ductile elements.

Ductility of members is achieved by using reinforcement with sufficient ability to deform beyond the elastic limit. For this reason,

***Table 3** Comparison of design base shear ratios between SANS 10160 (2009) and international standards for shear wall structures (soil Type 2)*

Building height (m) [storeys]	Structural type	No of walls	Design base shear ratio			
			SANS 10160 (2009)	UBC (1997)	EN 1998 (Type 1)	EN 1998 (Type 2)
13,2 m [4]	Shear wall (5 m)	2	0,088	0,092	0,076	0,050
		4	0,062	0,062	0,077	0,060
	Shear wall (7 m)	2	0,078	0,080	0,077	0,060
33 m [10]	Shear wall (6 m)	2	0,038	0,043	0,041	0,023
		4	0,046	0,044	0,049	0,032
66 m [20]	Shear wall (7 m)	4	0,028	0,027	0,035	0,018

***Table 4** Behaviour factors from different standards*

System	SABS 0160 (1989)	SANS 10160 (2009)	UBC (1997)	Eurocode 8 (2004)(1)	Swiss Code 262 (2003)(2)
Reinforced concrete shear walls	5	5	5,5	3 (medium ductility) 4 (high ductility)	4
Concrete moment resisting frame	2	3	3,5	3 (medium ductility) 4,5 (high ductility)	4
Steel moment resisting frame	5	4,5	4,5	4 (medium ductility) 5 (high ductility)	–
Non-ductile structures	1	1	–	1,5	2,0

Notes:

1 Provision is made for the multiplication of these values by a factor ranging between 1,0 and 1,5.

2 Values presented are those for reinforcement steel with a ratio of > 1,15 between the tensile and yield strengths.

design standards specify a required ratio between the tensile strength and the yield strength of reinforcement as well as a fracture deformation limit. South African reinforcement is required to have a ratio of at least 1,15 between the tensile and yield strengths (SANS 920 2005). The requirement from the ACI 318 (2002) is that this ratio should not be less than 1,25. The Swiss Code (SIA 262 2003) distinguishes between different steels with the ratio between yield strength and tensile strength varying between 1,05 and 1,15. The stress-strain behaviour of South African reinforcement steels needs to be verified for compliance with the ductility requirements of Eurocode 8 (2004).

Table 4 gives an extract of the behaviour factors in SANS 10160 (2009). It also provides a comparison with the behaviour factors in Eurocode 8 (2004), UBC (1997) and the Swiss Code (SIA 262, 2003). The choice of 5,0 for reinforced concrete shear walls is not consistent with Eurocode 8 (2004).

The behaviour factor for reinforced concrete *moment resisting frames* used in SANS 10160 (2009) is similar to that in Eurocode 8 (2004), justified by the improved detailing rules in SANS 10160 (2009). Although Eurocode 8 (2004) and the Swiss Code (SIA 262, 2003) use lower values for *reinforced concrete shear walls*, a decision was made by the seismic working group not to reduce the behaviour factor from the value in SABS 0160 (1989). The introduction of detailing rules, which goes beyond the requirements of SABS 0160 (1989), does not merit a reduction in behaviour factors at the same time. For this reason the value for SANS 10160 (2009) is taken to be the same as that in SABS 0160 (1989). The value of 1,0 has been retained for structures required to remain elastic (as opposed to the Eurocode 8 value of 1,5). The above comparisons and reasoning show that there is a need for local research to determine an appropriate behaviour factor value for South African materials, building practice and seismic input as defined in the local standard.

Design methods

SABS 0160 (1989) specifies the equivalent lateral static force (ELSF) procedure as a design method. No other design method is mentioned, and no limitations are set for the use of this method. The result is that structures can potentially be designed on the basis of this method, regardless of structural concept, possible irregularities in elevation or plan or other unfavourable characteristics.

In contrast, Eurocode 8 (2004), UBC (1997), SIA 261(2003) and NZS (1992) provide clear limitations on the use of the ELSF procedure. These standards all require dynamic analyses either in the form of a modal response spectra method or a time history analysis for those structures that do **not** meet the limiting criteria for the use of the ELSF method. The requirements for the ELSF method relate to those buildings in which the dynamic response is not significantly affected by contributions from higher modes of vibration. This would be the case for buildings which comply with the following characteristics:

- the building has regularity in plan and elevation
- the structure is limited in height
- the magnitude of the fundamental period of vibration of the building is within certain defined limits.

The UBC (1997) allows the use of the ELSF procedure for some categories of buildings in seismic zones with low nominal peak ground accelerations, regardless of regularity or fundamental period of vibration. Such an allowance is not made in the more recent Eurocode 8 (2004) and SIA 261 (2003).

In SANS 10160 (2009) the ELSF method is the only analysis method provided (as before). The limitations of the ELSF procedure are now clearly defined in Eurocode 8 (2004). For buildings outside the scope of these limitations, designers are referred to specialist literature for other more advanced design methods, which can include response spectra method, non-linear time history analysis and others. The purpose of not providing details for alternative design methods is to force designers to follow acceptable conceptual principles for buildings in seismic zones. It is also an attempt to prevent designers without the necessary knowledge and training from blindly using available software packages where such possibilities are offered.

The method provided in SANS 10160 (2009) for distribution of the total base shear over the height of the building is similar to the methods in international standards. It differs from the formula in SABS 0160 (1989) by the omission of some power factors in accordance with Eurocode 8 (2004).

Calculation of the first natural period

The ELSF procedure requires the calculation of the first natural period of the structure. Similar formulae for calculating the period are used in Eurocode 8 (2004) and the UBC (1997). The simplified formulae for steel-framed structures and for *moment resisting frames* in SABS (1989) have not changed in Eurocode 8 (2004) and the UBC (1997) since SABS 0160 (1989) was issued.

The formula for the first natural period of *shear wall structures* has now been adopted from Eurocode 8 (2004) (similar in the UBC (1997)). The formula takes into consideration the length of the wall and the surface area of the building footprint. This approach seems to be more logical than the formula in the SABS (1989) where the first natural period is only a function of the building length (and not of the wall, which essentially provides the stiffness). It is, however, pointed out by Goel & Chopra (1998) that these simplified formulae can be very conservative. The first natural period (T) for buildings up to 40 m in height is given by:

$$T = C_T \,.\, h_t^{3/4} \tag{3}$$

where, according to Eurocode 8 (2004), C_T is 0,085 for steel frames, 0,075 for reinforced concrete moment-resisting frames and for eccentrically braced frames and 0,05 for all other buildings; and h_t is the height of the building (in m) from the foundation or from the top of a rigid basement.

Alternatively, for structures with concrete or masonry shear walls (Eurocode 8 (2004)), the value of C_T may be taken as:

$$C_T = \frac{0{,}075}{\sqrt{A_c}} \tag{4}$$

where

$$A_c = \sum[(A_i \,.\, (0{,}2 + \frac{l_{wi}}{h_t})^2)] \tag{5}$$

A_c is the total effective area of shear walls in the first storey of the building (m²) (subject to the walls remaining relatively unchanged over the height of the building); A_i is the effective cross-sectional area of the shear wall i in the first storey of the building (m²); h_t is as above; and l_{wi} is the length of the shear wall i in the first storey in the direction parallel to the applied forces (m), with the restriction that l_{wi}/h_t shall not exceed 0,9.

Although not part of the formulation in the standard, if the designer wishes to use a simplified method, a more accurate method of determining the first natural period rather than using the formulae above would be to use the Rayleigh method (Eurocode 8 (2004), SIA 261(2003), UBC (2007)) which is not presented in SANS 10160 (2009). It is, however, generally found that the first natural period of a structure is best determined by a proper dynamic analysis of a structure. SANS 10160 (2009) gives guidance for calculation of the fundamental period when using a properly substantiated analysis. Among others, it is stated that the Young's modulus of elasticity of concrete should be taken as 0,5 times the short-term value for concrete to allow for the effect of cracking. In such an analysis the possible stiffening effect of finishes and infills may have to be considered as well. For this reason, the standard requires that the value of the first natural period calculated by alternative methods should not be more than 1,4 T as obtained from Equation (3) (although not present in Eurocode 8 (2004), this restriction is taken from the UBC (1997)).

Other revisions

Other relevant revisions in SANS 10160 (2009) not discussed in this paper include :

- Importance factor for buildings
- Direction of the seismic action
- Displacement verification
- Effect of structural and non-structural masonry infill panels
- Normative annexure on detailing of reinforced concrete elements
- Normative annexure on un-reinforced load-bearing masonry

COMPARISON WITH OTHER STANDARDS

A comparison was made between the design base shear values calculated using SANS 10160 (2009) and other standards. For the format of presentation the design base shear is expressed as a ratio of the total seismic weight of the building (design base shear ratio).

Table 3 (shear walls) and Table 5 (moment frames) give a comparison between SANS 1060 (2009), the UBC (1997) and Eurocode 8 (2004). Comparisons are made with both response spectra types (1 and 2) of Eurocode 8. The comparison with the Type 2 spectra of Eurocode 8 is presented to demonstrate the large difference between the two spectra, Types 1 and 2. It provides the motivation for choosing the Type 1 spectra for SANS 10160 (2009) on the grounds of a more conservative approach.

***Table 5** Comparison of design base shear ratios between SANS 10160 (2009) and international standards for moment frame structures (soil Type 2)*

Building height (m) [storeys]	Structural type	No of cols	Design base shear ratio			
			SANS 10160 (2009)	UBC (1997)	EN 1998 (Type 1)	EN 1998 (Type 2)
13,2 m [4]	Moment frame	40	0,118	0,096	0,076	0,043
33 m [10]	Moment frame	40	0,058	0,054	0,044	0,025
66 m [20]	Moment frame	40	0,035	0,032	0,026	0,010

***Table 6** Comparison of design base shear ratios between SANS 10160 (2009) and SABS 0160 (1989) for 4-storey buildings*

Building height (m) [storeys]	Structural type	Building length (m)	Soil type (2008/1989)	Design base shear ratio	
				SANS 10160 (2009)	SABS 0160 (1989)
13,3 m [4]	Shear wall 4 x 5 m	35	1/S1	0,060	0,080
13,2 m [4]	Shear wall 4 x 5 m	35	2/S2	0,072	0,080
		35	3/S2	0,069	0,080
	Moment frame	35	1/S1	0,078	0,195
		35	2/S2	0,118	0,200
		35	3/S2	0,115	0,200

Values in the tables are presented for soil Type 2 of SANS 10160 (2009), which is the same as type B in Eurocode 8 and comparable to type S_c in the UBC. For each of these standards the load factor for seismic loads is unity, therefore a direct comparison is possible. The values are all presented for a nominal peak ground acceleration of 0,1 g.

Different structural heights and wall lengths were considered. Other variables are identified in the tables. From Table 3 it can be seen that values calculated for *shear wall structures* using SANS 10160 (2009) compare well with those of the other standards, except for the Type 2 response spectra of Eurocode 8. Table 5 shows that the SANS 10160 (2009) values for *moment resisting frames* are higher than the values of the other standards for low structures, but similar for medium to high structures.

A comparison is also presented between SABS 0160 (1989) and SANS 10160 (2009). Table 6 (4-storey), Table 7 (10-storey) and Table 8 (20-storey) present comparisons of a variety of parameters including the structural system. It can be seen that the values from SANS 10160 (2009) are significantly lower than those using SABS 0160 (1989), mostly due to the load factor of 1,6 used by SABS 0160 (1989) and the revised response spectra which have a more notable influence on taller structures. The values are presented for a nominal peak ground acceleration of 0,1 g.

CONCLUSION AND SUMMARY

This paper presents a critical overview of the background to proposed revisions on seismic loading in SANS 10160 (2009). The paper also presents a comparison to demonstrate the calibration of the proposed formulation against other standards.

Eurocode 8 (2004) was in principle used as the reference for the formulation of the revised clauses. Other international standards are referenced for comparison. By using the universally acceptable approach of considering seismic events in a manner similar to an accidental action, the load combination factors for seismic action has been substantially revised. The formulation for seismic design in SANS 10160 (2009) is now based on the concept of regular buildings with suitable detailing to allow the necessary ductile behaviour. Guidelines for suitable detailing are now included in the standard.

In the proposed SANS 10160 (2009) a redundancy factor is used to compensate for the choice of the design peak ground acceleration and the behaviour factors for reinforced concrete shear walls. It is suggested in the paper that the use of this redundancy factor be reconsidered and that appropriate

***Table 7** Comparison of design base shear ratios between SANS 10160 (2009) and SABS 0160 (1989) for 10-storey buildings*

Building height (m) [storeys]	Structural type	Building length (m)	Soil type (2008/1989)	Design base shear ratio	
				SANS 10160 (2009)	SABS 0160 (1989)
33 m [10]	Shear wall 4 x 6 m	35	1/S1	0,030	0,069
		35	2/S2	0,046	0,080
		35	3/S2	0,052	0,080
	Moment frame	35	1/S1	0,039	0,123
		35	2/S2	0,058	0,161
		35	3/S2	0,067	0,161

***Table 8** Comparison of design base shear ratios between SANS 10160 (2009) and SABS 0160 (1989) for 20-storey buildings*

Building height (m) [storeys]	Structural type	Building length (m)	Soil type (2008/1989)	Design base shear ratio	
				SANS 10160 (2009)	SABS 0160 (1989)
66 m [20]	Shear wall 4 x 7 m	35	1/S1	0,018	0,043
		35	2/S2	0,028	0,057
		35	3/S2	0,032	0,057
	Moment frame	35	1/S1	0,023	0,087
		35	2/S2	0,035	0,114
		35	3/S2	0,040	0,114

values be defined for the peak ground acceleration and the shear wall behaviour factor.

The damage limitation criteria has been taken from the UBC (1997) and should be based on the Eurocode 8 (2004) requirement to be consistent with the formulation of the standard.

It is shown that the revised formulation provides lower design forces for regular buildings than in SABS 0160 (1989). The current available information on nominal peak ground acceleration is presented, and it is pointed out that a decision is needed for the choice of this important parameter for seismic zones in South Africa.

Further research is also needed to determine a suitable behaviour factor for concrete shear walls in South Africa. This value is taken from SABS 0160 (1989). It is larger than that of Eurocode 8 (2004) and therefore less conservative.

The sections in the standard covering mining-induced seismicity have not changed. It is suggested that response spectra be defined for the design of structures in these regions.

The South African industry has had limited exposure to the design of building structures for seismic loading. It is important that the revised standard and the correct design and construction procedures be adopted for all designs in regions of seismicity. For this purpose, an awareness and training programme is required for the local profession. The revised standard aims to introduce the notion that correct conceptual layout and appropriate detailing of plastic zones in a building will go a long way in providing seismically resistant structures in areas of moderate seismicity.

ACKNOWLEDGEMENTS

The contributions by the following individuals who spent many hours on the compilation of the revised standard are acknowledged: Mr Keith Bokelman (Aurecon), Mr Frik Riedeman (Liebstan), Mr Colin Roux (Sutherland Consulting Engineers), Mr Ferdie Heyman (De Villiers Partnership) and Mr Alten Hulme (Hulme Consulting Engineers). The valuable comments from Prof Alessandro Dazio of the Swiss Federal Institute of Technology in Zurich are highly appreciated.

REFERENCES

ACI 318-02 2002. Building Code Requirements for Structural Concrete. American Concrete Institute.

ATC 3-06 1978. Tentative Provisions for the Development of Seismic Regulations for Buildings. Applied Technology Council, NBS Special Publication 510, US National Science Foundation.

Booth, E D, Kappos, A J & Park, R. 1998. A critical review of international practice on seismic design of reinforced concrete buildings. *Journal of Structural Engineering*, 76(11).

Chopra, A 2002. *Dynamics of Structures*, 2nd ed. New York: Prentice Hall.

Dazio, A 2007. Short course: Seismic Design of Building Structures. ETH Zurich. *Earthquake Hazard Centre Newsletter* 1999. 3(1): 4, July.

Eurocode O/BS EN 1990. Basis of Design. European Committee for Standardization, issued April 2002.

Eurocode 1 EN 1991. Actions on Structures. European Committee for Standardization, issued April 2001.

Eurocode 8 EN 1998-1 Design of Structures for Earthquake Resistance Part 1: General rules, seismic actions, and rules for buildings. European Committee for Standardization, issued December 2004.

Goel, R & Chopra, K 1998. Period Formulas for Concrete Shear Wall Buildings. *American Society of Civil Engineers, Journal of Structural Engineering*, 124, April.

Historical Earthquakes 2007. Council for Geoscience, Pretoria, Available at: http://196.33.85.14/cgs_inter/content/view/307/299/.

Kijko, A, Graham, G, Bejaichund, M, Roblin, D L & Brandt, M B C. 2003. Probabilistic peak ground acceleration and spectral seismic hazard maps for South Africa. Report No. 2003-0053, Council for Geoscience, Pretoria.

Milford, R V & Wium D J W 1991. The impact of seismic events on buildings in mining areas. *The Civil Engineer in South Africa*, 33(12).

NZS 4203 1992. New Zealand Standard Code of Practice for General Structural Design and Design Loading for Buildings. Volume 1. Code of Practice. Standards New Zealand.

Paulay, T & Priestley, M J N 1992. *Seismic Design of Reinforced Concrete and Masonry Buildings*. New York: Wiley Interscience.

Retief, JV & Dunaiski P E 2008. Update on SANS 10160 (Draft). *Civil Engineering*, 16(4).

SABS 0160-1989 Rev. 1993. South African National Standard. Code of Practice: General Procedures and Loadings to be Adopted in the Design of Buildings. Pretoria: South African Bureau of Standards.

SANS 10160 2009 (Draft standard). Basis of Structural Design and Actions for Buildings and Industrial structures. Pretoria: South African Bureau of Standards.

SIA 261 2003. Actions on Structures. Swiss Society of Engineers and Architects, Zurich: Swiss Standard 505 261.

SIA 262 2003. Concrete Structures. Swiss Society of Engineers and Architects, Zurich: Swiss Standard 505 262.

UBC (Uniform Building Code) 1997. International Conference of Building Officials, Witthier, California.