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The Dudley and Manchester UK Earthquakes

Contents

Put into perspective by **Alice Walker** and **Chris Browitt** of the British Geological Survey

World Earthquakes

Earthquakes can occur anywhere in the world, although they are not uniformly distributed, with the majority at boundaries of the great plates that make up the outer skin of the earth and which move at about the speed our fingernails grow, driven from below. Globally, there are around 800 'moderate' earthquakes, (magnitude 5 to 5.9 Ms), 120 'strong' ones (magnitude 6 to 6.9) and around 20 'major' earthquakes, of magnitude 7 or greater, each year. There are many more smaller ones; some 70,000 reported internationally in 2001, but most were unknown except to the seismologists who study them. The main hazards during and following a larger earthquake include ground shaking, landslides, tsunamis and ground liquefaction. Fires may rage due to ruptured gas or water mains, and access for emergency services may be blocked. The great fire in San Francisco following the 1906 earthquake, lasted three days and was more damaging than the shaking itself. Firestorms after the 1923 Tokyo earthquake, killed over 38,000 people. Recent fatal earthquakes in El Salvador and India, on 13 January and 26 January, 2001 (both magnitude 7.7 Mw) killed 800 and 20,000 people, respectively.

The Italian earthquake on 31 October 2002, which killed 26 schoolchildren and teachers, was of modest size; with a

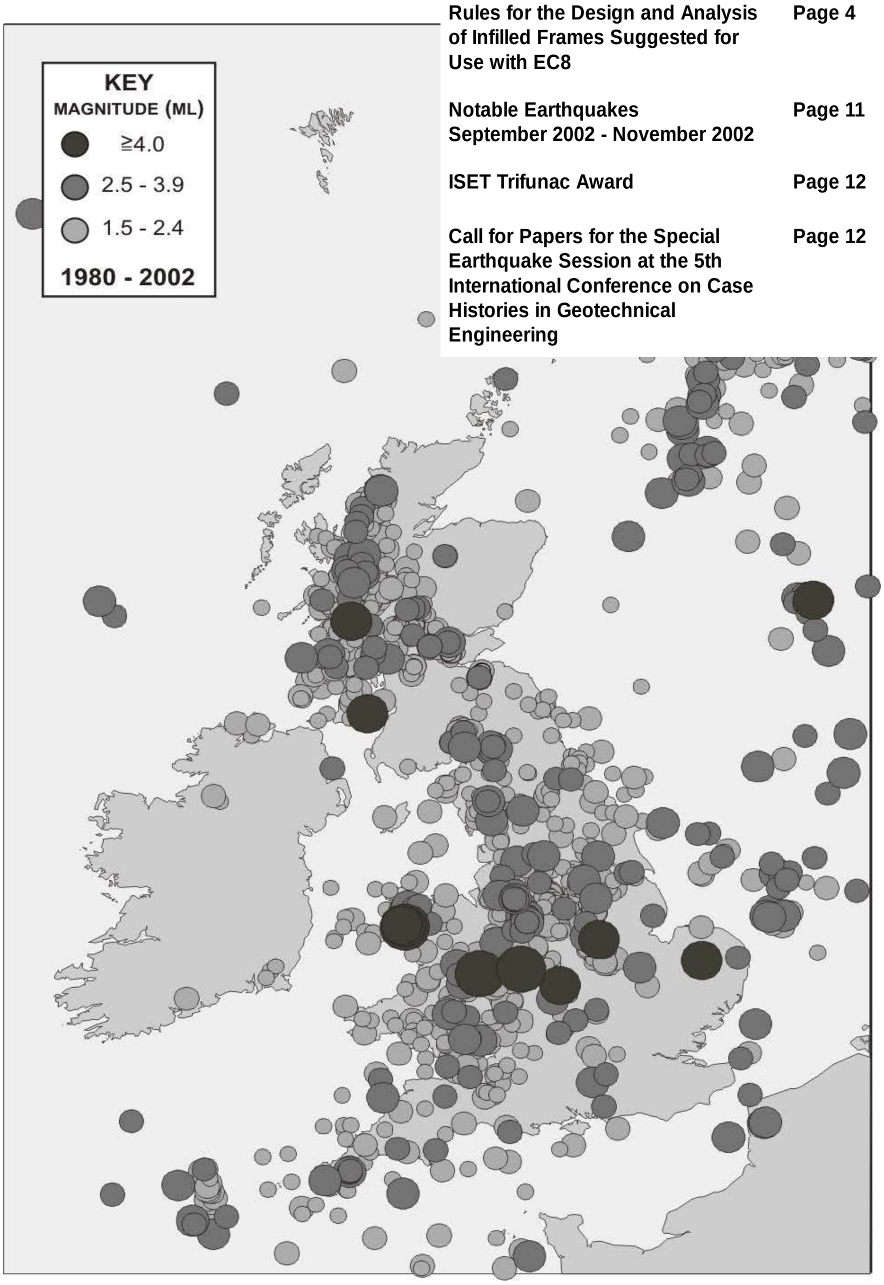


Figure 1. Epicentres of earthquakes with magnitudes 1.5ML or greater, for the period 1980 to November 2002

Figure 2. Damage to a chimney in the Bloxwich area from the Dudley Earthquake

magnitude of 5.6 Ms, 5.9 Mw; there are about 270 of this size or greater each year, worldwide. At BGS, it was realised within an hour or so (as soon as the magnitude was calculated) that this earthquake was too small to have caused any well-constructed buildings to collapse, as was being suggested in early reports. TV images the next day showed the extent of the destruction of the local school and the minimal damage elsewhere, clearly demonstrating poor quality construction and revealing a tragedy which should never have happened.

UK Earthquakes

In the UK, we are not immune from earthquakes experiencing around 200 each year with about 20 felt by local residents (Fig 1). The largest, in 1931, to affect the United Kingdom was centred on the Dogger Bank; fortunately, 100 km out in the North Sea. It had a magnitude of 6.1 ML and caused minor damage on the east coast of England where many chimneys fell down. Onshore, the largest earthquake in the last 140 years, occurred in North Wales on 19 July 1984 with a magnitude of 5.4 ML. It was felt over most of England, throughout Wales and even into Scotland and Ireland. It caused some damage as far as Liverpool, 120 km from its epicentre. More recently, an earthquake with a magnitude of 4.2 ML near Warwick on 23 September 2000 and another near Melton Mowbray in October 2001 (magnitude 4.1 ML) were felt over much of England and Wales. There were many reports of objects such as ornaments, pictures or toys falling or being displaced. In a few cases, heavy objects, including washing machines, cookers and lounge furniture were also said to have moved, but no damage was reported.

The Dudley Earthquake

The third large earthquake to strike central England in the space of 2 years was centred on Dudley, in the West Midlands, on 23 September 2002, with the larger magnitude of 4.8 ML (8-11 times bigger in energy than the previous two). Again, people were awakened over a wide area and the felt effects stretched from Dublin, Ireland to the east coast of England and from Yorkshire to the south coast and Devon. In the epicentral area, there was much alarm and some damage to chimneys and roofs (Fig 2), with plaster cracking on interior walls, indicating a maximum intensity of 6 on the European Macroseismic Scale (which describes the degree of shaking in an earthquake). Information on these effects has been gathered through some 8000 responses to BGS questionnaires distributed nationwide through the media and internet. Some typical felt reports were “I woke up frightened and clinging to my bed! I thought that the roof was going to come down on me, I didn’t realise that it was an earthquake but there seemed to be a lot of noise above me in the loft and some banging”; “The bottles on our shelf started rattling violently, and then the whole room started moving from side to side”; “I was sitting at my computer when the whole house started to shake violently. I could see walls and ceiling moving also kitchen wall cupboards. My computer screen was shaking and I could feel the floor heaving beneath me”; “I was asleep and the whole house (located on top of a hill) shook for over 10 seconds. There was a deep disturbing rumbling/rattling noise all around me and I felt I was lying on top of a large oscillating jelly structure. I ran out the house to check what was happening but everything seemed normal and quiet”; “the whole of the house was shaken and my glass full of water smashed as it fell off the table”. These descriptions are typical for larger earthquakes in the UK and for some smaller ones in the epicentral area. So far, there have been two reports of people injuring themselves as a result of rushing out of the house in alarm – one person broke their leg falling down stairs, another broke their toe. A map showing the felt area of the Dudley earthquake is given in Figure 3.

Shock waves from the Dudley earthquake were recorded across the UK on the BGS seismic monitoring network and throughout Europe. A seismogram of the ground movement and the different seismic waves recorded is shown in Figure 4. This was the largest earthquake to affect the UK since a magnitude 5.1 ML event near Shrewsbury in 1990 which was felt throughout England and Wales and caused some damage near its epicentre.

To put the Dudley earthquake into perspective: 1,300 earthquakes of this size or bigger occur each year somewhere in the world. However, in the UK we expect, on average, an earthquake of this size or bigger to occur once every eight years. So we might think that it will be another 8 years before the next large one, but statistics don’t work like that, and the next 4.8 magnitude earthquake could occur tomorrow, next year or next century.

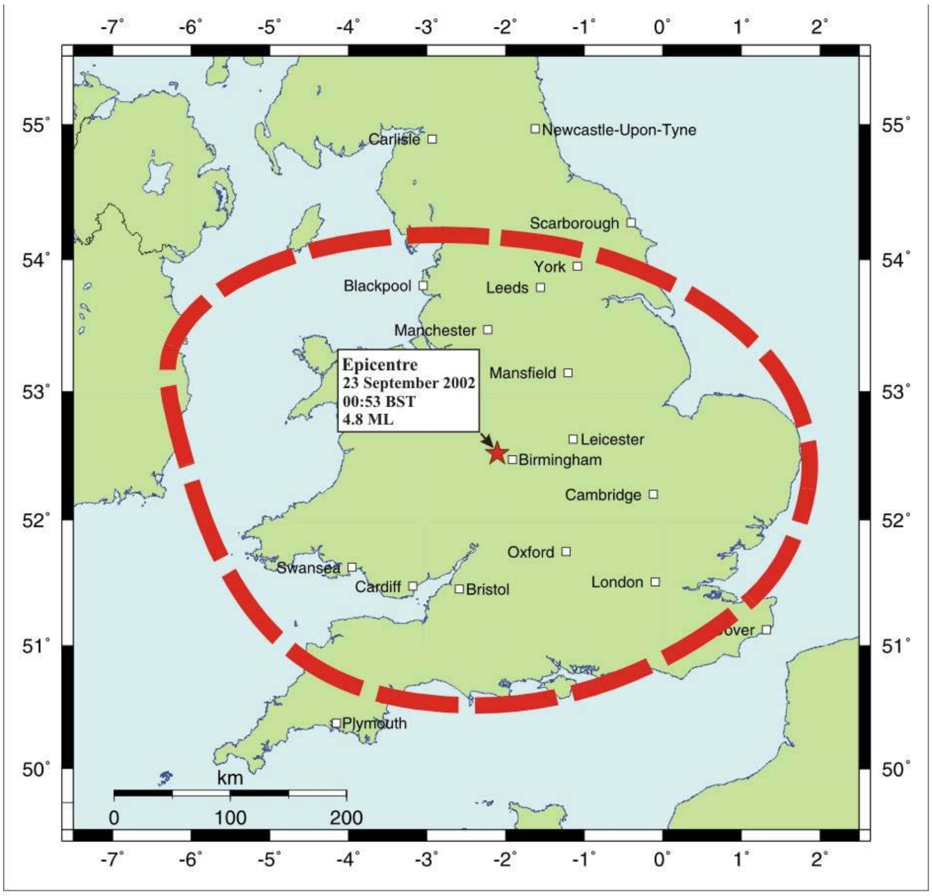


Figure 3. Felt area of the Dudley earthquake, 23 September 2002, 00:53 BST, 4.8ML

Seismologists have not yet solved the problem of predicting earthquakes but with increasing objective data being collected they are constantly improving their assessments of how likely earthquakes are, and are able to inform engineers and planners accordingly.

Details of British and important global earthquakes are posted on the BGS seismology web site which is continually updated. During the Dudley earthquake some 360,000 hits were received on the day it occurred indicating the power of this medium for disseminating objective information.

The Manchester Earthquakes

Most recently, there has been a sequence of earthquakes in the Manchester area, with the highest magnitude being 3.9 ML. In the 5 weeks following this strongest shock on 21 October, some 106 earthquakes, with magnitudes between 1.1 and 3.9 have been located, with 36 reported to be felt. There has been no significant damage and most of the felt reports have been confined to the greater Manchester area, with a number of reports received up to 30 km away for the largest event. Some typical ones were "The whole building wobbled and shook producing a sensation of being disoriented" and "I could feel the ground shaking below my feet. Desks in the classroom were vibrating as well". These indicate that the maximum intensity experienced was 5 EMS (European Macroseismic Scale). Immediately after the first felt earthquakes, a temporary network of three seismometers was installed in the epicentral region. The addition of these new stations permits the depths of the events to be established more accurately, with the emerging result that they are shallow, between 2 and 4 km. This evidence fits well with the felt effects (small earthquakes felt over restricted areas), and shows that this is one of the shallowest sequences in the UK. At the time of writing (29 November, 2002) there have been no reports of felt earthquakes in the past 10 days, indicating that the activity has started to decline.

The UK Seismic Network

Earthquakes, both globally and in the UK, are monitored using the BGS seismic network of 146 seismometer stations. Data is transferred to Edinburgh four times a day (or on demand during periods of particular interest) using either dial-up telephone lines or the public internet. Within 1 to 2 hours, the location, magnitude and nature of an event (e.g. earthquake, explosion, sonic boom, or mining-induced seismicity) are determined and the results are widely disseminated. Interest from BGS' wide spectrum of customers in government, industry and academia, and from the media and the public is often intense. A 24-hour on-call service is operated, with computer connections between staff members' homes and the BGS Edinburgh office allowing rapid analysis. For more information visit www.earthquakes.bgs.ac.uk.

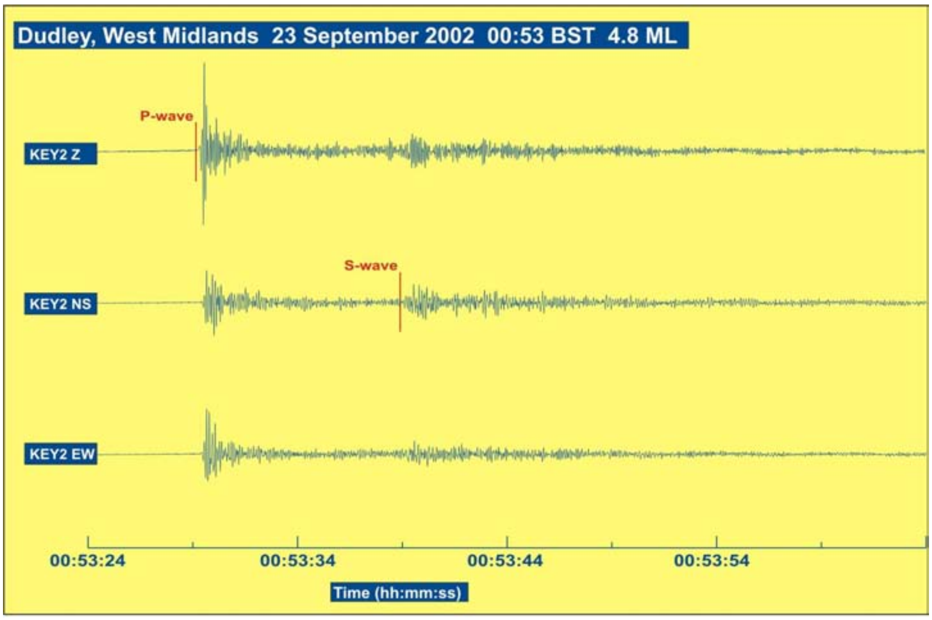


Figure 4. Seismograms recorded on the BGS seismograph station at Keyworth, near Nottingham, from the magnitude 4.8ML earthquake of 23 September 2002 00:53 BST

RULES FOR THE DESIGN AND ANALYSIS OF INFILLED FRAMES SUGGESTED FOR USE WITH EUROCODE 8

By **David GE Smith** of Scott Wilson

1. Infilled Frames and EC 8

Earlier drafts of Eurocode 8 have given rules for the design of masonry infilled frames of reinforced concrete or structural steel in which the infills are in contact with the frames, but free to distort in a shear mode. The surrounding frame was assumed to distort into the usual frame racking mode. These rules were unduly conservative, but nevertheless indicated how the infills could be represented in the structural analysis. This was not unreasonable at the time, since infills were then considered by many to have been responsible for extensive damage to commercial and residential properties during earthquakes in many countries. It is now acknowledged that, with a few exceptions in which the masonry panels were poorly distributed within the building, the damage would have occurred in the absence of the infills due to the inadequacy of the frame. In fact, without the infills, the damage would usually have been far greater. The structures concerned were invariably non-ductile moment resisting frames up to four storeys in height. In many, the columns are no wider than the masonry panels and too thin to accommodate effective confining reinforcement. Indeed, the panels had been responsible for the failure of the frames, since they had imposed shear forces upon the joints in the frame far in excess of their low capacity.

The Final PT Draft of Eurocode 8 (Stage 34) [1] takes account of the new perspective on infilled frames adopted since the mid-1990s, following an assessment of their performance during the Northridge earthquake and an experimental and theoretical investigation[2]. There is, despite this investigation, as yet, no consensus of view as how to represent the infills in the structural analysis and in the design, matters which are both deferred to the National Annex. This situation is unsatisfactory at such a late stage in the development of a Standard, due to the number of low rise structures affected and by the number of European countries (the majority, if not all!) who have no rules. There is however useful guidance in the final draft on the panels and on the disposition in the structure as follows:
(a) A calculated reduction in the behaviour factor to cover the common situation when less bays are infilled at ground floor than above.
(b) A requirement to consider the distribution of infills in plan. Where the lack of uniformity is considered sufficient to increase the torsional response, there is a subjective increase in the additional eccentricity for cases of moderately irregular (a subjective classification) distributions and a potentially time-consuming analytical study for more irregular distributions.

A procedure is defined in 4 below which removes the more arbitrary aspects of (b).

2. The Wider Perspective and the Position in the UK.

In post war years large experimental programmes into infilled frames were conducted in the UK [3-5], the Soviet Union [6], Mexico [7,8] and the United States [9-11] and a large number of countries had smaller programmes. The Russian tests included pinned frame connections, a wide range of mortars and both solid and perforated blocks. The Mexican tests included a wide range of types of masonry, including various forms of hollow blocks and very weak frames. The American tests included multi-bay frames. A high proportion of these were on full scale specimens and many simulated reversible and repeated loading. In the UK two design approaches were developed, one an elastic method which gave good forecasts of the serviceability and ultimate strengths of the masonry panel and giving the effects on the frame [12,13], and the other [14] giving the failure load of the combined frame infill system. The former method has been further developed by Dr RH Riddington of Sussex University, though this work is unpublished.

In the 1970s a draft code for the design of composite construction [13] was prepared in parallel with the draft of the future BS5950 : Part 1 (the British Standard for design of rolled and welded sections), which was more comprehensive than the eventual BS5950 Part 3.1 [8] (Composite Construction – Simply supported beams). The draft included continuous beam construction, composite columns and frames and infilled frames, all topics beyond the scope of the present Standard. The rules for infilled frames (which was restricted to steel frames) had been included because they were perceived as being a method of resisting lateral loading from wind and for preventing progressive collapse. In fact, three series of large scale tests were undertaken to support investigations into the stability of systems built flats. This followed the Ronan Point disaster, where progressive collapse had occurred as a result of a gaseous explosion in a corner room high up in a block of flats of an imported form of building construction, which it appears had escaped assessment by UK engineers. These tests included RC frames.

In recent years shake table investigations have been undertaken, including recent shake table tests in the UK and in Italy[2].

3. Proposals to Satisfy the Requirements of UK Engineers

Users of Eurocode 8 in the UK are unlikely to be involved in the design of the type of infilled frames described in 1 above, since such structures are normally designed by local engineers in the country concerned. They are however much more likely to be involved with the assessment of such structures in various forms. For this reason the rules in Ref 12 have been modified to include steel and RC frames, continuous and 'pinned' frame connections, masonry and concrete infills, other forms of confined masonry considered in Eurocode 8, Part 1, gaps around panels and holes.

The modifications have been derived by assessing 700 test results using the methods in Refs 12 and 14, selecting the method (the updated version of Ref 12) which gave the best overall forecast for the standard case of continuous frames completely filled by the panels and calibrating it against the relevant conditions. They give a reasonable representation of the in-plane performance, both in regards to stiffness and strength, of the shake table tests on infilled frames reported in Ref 2. The rules also allow for the difference in the compressive strength of masonry in the vertical and horizontal directions, which is significant with some hollow blocks. They are tolerant of small gaps between the frame and the infill, but the rules are modified where the effect is likely to be significant. The main problem with gaps is that they encourage panels to fail out-of-plane, a topic investigated recently at the University of Bristol[2].

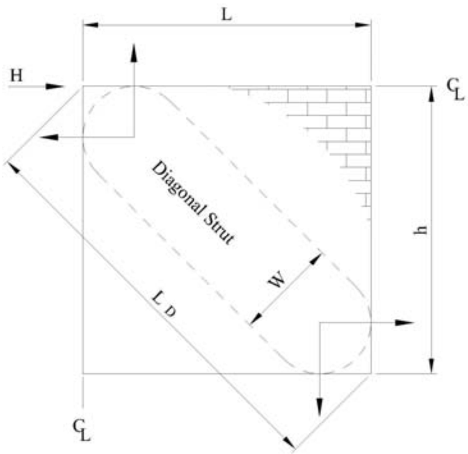
The method is based upon a realistic model of the panel with a diagonal compression force through the panel. The compression diagonal in imperforate masonry panels is subject to concentrated compression forces at the ends. The width of the diagonal is determined by the relative compressive stiffness of the masonry, the flexural stiffness of the frame members and the joint rigidity between the members of the frame. The width of the strut at the ends controls the compression strength of the panel (if imperforate) and results in moments and shear forces being applied to both the columns and the beams.

In the corner of concrete infills, diagonal splitting occurs, but this would appear to be a failure condition only with frames of low stiffness. Sliding occurs on the horizontal bedding planes of masonry infills, where the vertical component of the compression force increases the sliding resistance. The British National Application Document to Eurocode 6 reproduces the sliding resistance expression in BS5628, which gives appreciably higher sliding resistances than many National standards. Some other users of Eurocode 8 are obliged, in their implemented versions of Eurocode 6, to use appreciably lower sliding resistances. Sliding on bedding planes however is not a failure condition, except possibly with masonry units which are exceptionally fragile and particularly prone to earthquake damage, in particular calcium silicate bricks (banned by some specifying authorities in the USA and the UK) and some hollow blocks. For other infill materials, sliding can be disregarded under the 500 year earthquake. It should however be included when considering the earthquake likely to occur within the design life of a structure, since the repair of badly cracked masonry is expensive.

The method is presented as an Annex L to Eurocode 8 : Part 3 [15], which is to replace Eurocode 8 : Part 1.4. It can be used in two ways. In Ref 13, the horizontal force was divided amongst the panels according to the rules of statics, but disregarding one panel in four. This was considered to be a reasonable allowance for minor building alteration or major damage. A panel stiffness was provided to enable building deflections to be estimated, which is suitable for the determination of the period of vibration. An earlier version of the expression (the only published version) has been shown to be suitable for use in time-history or pushover analysis[2].To the frame shears and moments from the analysis, must be added the local moments and shears at the ends of the diagonal discussed in the preceding paragraphs.

With the method of applying material safety factors in the Eurocodes and British Standards, a problem arises in that the very different safety factors, applied to masonry, structural steel and reinforcement, results in the masonry being the weakest part in design, whereas in tests it may be the strongest part. As a result, the calculation model does not reflect the failure condition, implying an incorrect structural response. In this situation, the approach to material safety factors in American Codes (comparable to the materials safety factor in shear in concrete in BS8110 and Eurocode 2, which is approximately the RMS value of the safety factors for concrete and reinforcement) gives a better representation of the situation. The direct consequence of using the European approach to design is that the panel is stronger than is desirable for economic assessment of earthquake resistance of the structure. Compounding this with the design philosophy elsewhere in Eurocode 8, that the frame must be capable of resisting the likely diagonal compression resistance of the panel, a force is imposed on the frame far in excess of the force during the design earthquake. It is unreasonable to expect existing structures to be capable of resisting such forces and for this reason, in Annex L, a less onerous increase is specified, given by g_{rd} in Clause L3(1)b , which is applied to the horizontal excitation.

Annex L, appended, is presented in the same format as EC8 Part 3. As in the



Values of A, B & W in Figs 1 & 2

	Without windows	With windows
A	L/20	L/4
B	L/20	L/4
W	From L.2(4)	From L.2(4)

Figure 1. Diagonal Strut Representation of Masonry Infill Panel

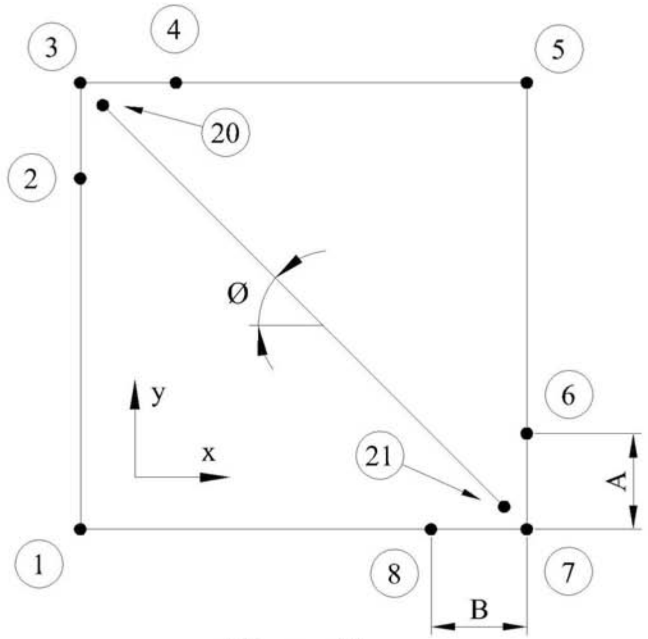


Figure 2a

Diagonal Strut Representation by Constraint Equations.

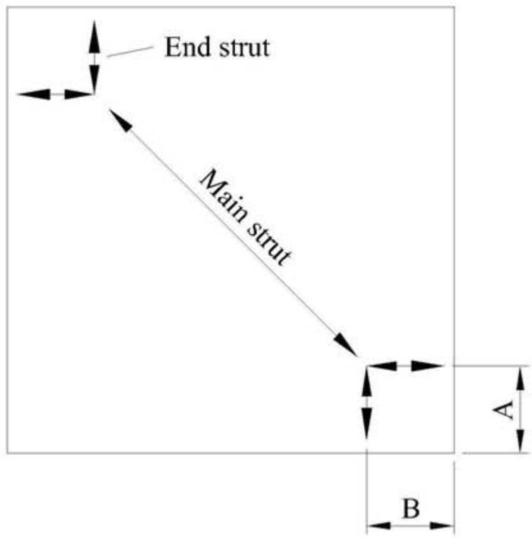


Figure 2b

Diagonal Represented by FE Strut System

The strut has nodes 20 & 21 at its ends, which are coincidental with notes 3 & 7 but not connected to them. Constraint equations ensure:-

$$\Delta^{20}_{y} = \Delta^{4}_{y} \quad , \quad \Delta^{20}_{x} = \Delta^{2}_{x}$$

$$\Delta^{21}_{y} = \Delta^{8}_{y} \quad , \quad \Delta^{21}_{x} = \Delta^{6}_{x}$$

The widths of the struts are:-

Main strut = W

End struts = W/2

Eurocodes, certain values are ‘boxed’; a device used here to enable specifying authorities to chose different values where they can be justified.

4. Representation of Masonry Infills in Analysis

The method of representing the infills in analysis, suggested in earlier drafts of Eurocode 8,was found unsatisfactory and has been omitted. Two models which approximately represent the effects on the frame specified in Annex L are given below. Both use the cross-sectional area and stiffness in Annex L, but the contact points are at the distances from the nodes of the frame to represent approximately the frame shears and frame moments defined in Annex L, as shown in Figure 1. These are one tenth of the panel length for imperforate panels or, where there are large openings, (beam length)/4 is substituted. It is important that the force is imposed transverse to the frame member (so there is no component of force along the member).

The simplest model uses constraint equations. Each diagonal extends between the opposite corners of the bounding frame, but is not connected to it. In time history analysis the member is removed when subject to tension. The displacements of the end nodes of the compression diagonals are made equal to those at the contact points of the surrounding frame, as indicated in Figure 2a.

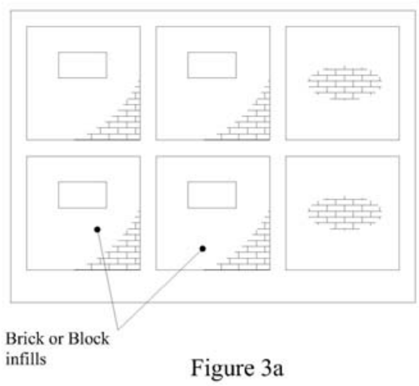


Figure 3a

Typical Infilled Bent.

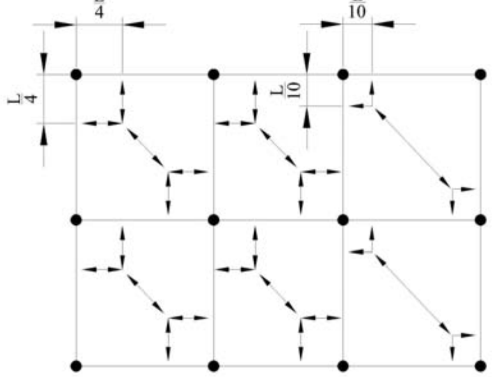


Figure 3b

Diagonal Strut Represented by FE Strut System

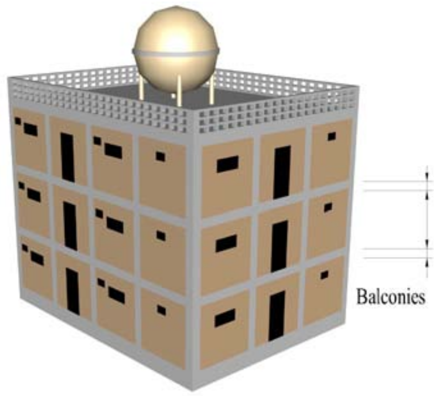


Figure 4a. Typical Villa in the Middle East

The alternative model replaces the compression diagonal by five struts, as shown in Figure 2b. In dynamic analysis, including spectral analysis, to avoid problems in the solution process, it is important that the struts are represented by members capable of carrying only axial forces (in most FE computer suites these are described as 'struts') and not by beam type members with zero flexural stiffness. The representation of the infills in the typical masonry infilled exterior wall shown in Figure 3a, is indicated in Figure 3b.

A typical Middle Eastern villa, which uses the type of wall system in Figure 3a, is illustrated in Figure 4a. Nearly all the masonry panels on the exterior have voids for windows, doors and ventilators. Ventilators can be disregarded if they satisfy the positioning rules in Annex L, Clause L.1(5)a. Panels with doors are entirely disregarded. Panels with the typical windows in such villas, which usually also have a ventilator, are nominally assigned a third of the stiffness of an imperforate panel. Weighting factors (k_2 in Annex L) can be assigned to the various panels in the analysis as shown in Figure 4b and the total number of effective panels so derived, as shown in Table 1. There are 5.33 effective panels in one direction and 6.67 in the other. The ratio between these is 1.25, which is reasonable. Where it is greater, additional infills in the weaker direction, or alternative measures, should be introduced to ensure the structure is of comparable strength in orthogonal directions.

Sometimes external columns, and particularly corner columns, will be subject to a net direct tension during larger excitations. This reduces the shear capacity where the columns are of RC. It is recommended such columns are designed using BS8110 and not Eurocode 2, or even the American Concrete Institute (ACI 318) /Uniform Building Code procedure. UK authorities consulted advise that the approach in BS8110 is the more justifiable and should be adopted, though they acknowledge there is scant research on this important topic. It is overlooked in the National Application Document for Eurocode 2 Part 1. For assessment, an optimistic view is sometimes taken, so the assessor here should make a judgement.

The situation where the distribution of the more perforated and less perforated panels is decidedly non-uniform is addressed in Eurocode 8 : Part 1[1], and it should be noted that, with the type of structure in Figure 4a there is often solid concrete blocks in the ground floor storey and weak hollow clays in the storey(s) above. There is then generally no need to allow for the type of vertical irregularity discussed in 1 above. Indeed sometimes the hollow block masonry is disregarded.

Sometimes engineers in the UK may be asked to assess the adequacy of

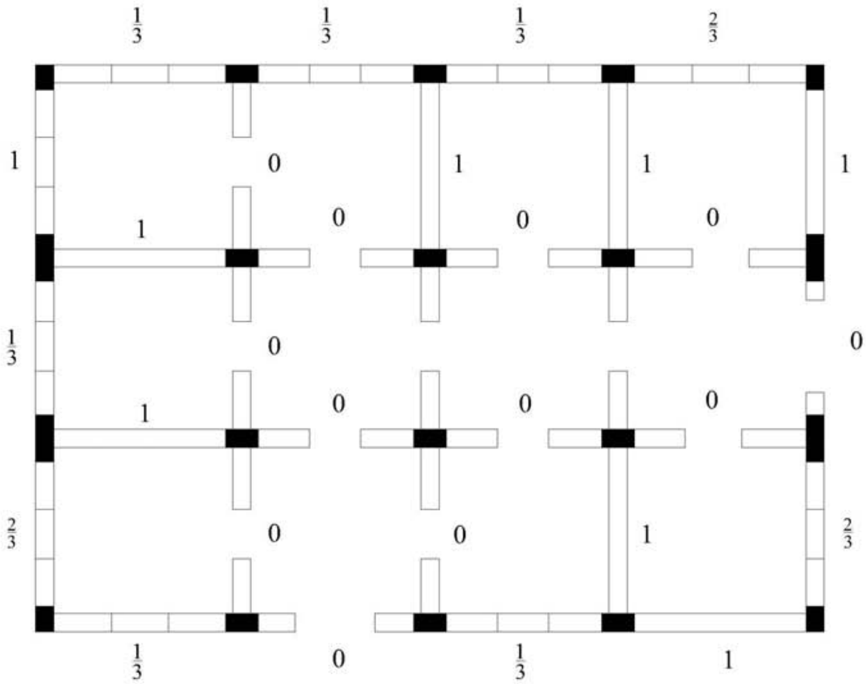


Figure 4b
Panel Weighting Factors

Table 1 : Effective Number of Panels

Direction	← →	↕
Weighting factor k_2	1 $\frac{2}{3}$ $\frac{1}{3}$	1 $\frac{2}{3}$ $\frac{1}{3}$
Number of panels Exterior Interior	 1 1 5 2	 2 2 3
Effective number of panels	$5\frac{1}{3}$	$6\frac{2}{3}$

an industrial steel building with masonry infilled panels, designed only for wind loading, for a UK earthquake with a ground acceleration of 0.25g. It is likely that, if assessed by the rules in Annex L, the beam/column connections will be found to fail in shear. Strengthening the connections in external walls may prove impracticable. If the buildings are being refurbished and new walls are required the connections are easily strengthened before infilling the panels.

5. Conclusions

Above is presented a comprehensive and workable set of rules for the design and analysis of infilled frames, which is suitable for lateral loading in the form of earthquake excitation, wind or blast and which does not require exceptional skill or task-specific software. Comments on these rules are invited.

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ANNEX L (Informative)
Masonry infilled frames subject to horizontal loading

L.1 Conditions

(1) Where reliance is placed upon framed panels to ensure the lateral stability of any storey in a frame, a sufficient number of such panels should be placed in that storey to allow alternative paths for horizontal loads. This is a provision for isolated panel collapse or the removal of some panels during future structural alterations. Where such deliberate removal could serve no useful purpose, it is sufficient to consider the simultaneous removal of one panel from each infilled bent in that storey, such as to produce the worst effects and disregard Clause 4.6.3.1(2) of prENV1998-1.

(2) The framing members should be of reinforced concrete complying with prENV1993-1 or of structural steel complying with prENV 1993-1 or of composite construction complying with prENV1994-1. The panel infills should be of masonry complying with prENV1996-1 or concrete infills complying with prENV1992-1 and with the appropriate modifications to the framing members in prENV1998-1 to justify the behaviour factors adopted in the justification.

(3) The height to length ratio of the panel should lie within the range [0.3 to 3.0].

(4) The out of plane aspect ratio of the panel, defined as the lesser of the vertical or horizontal dimension of the panel, or of compartment panels, to the effective thickness, shall not exceed [15], where:

- A compartment panel is a panel stiffened by surrounding cross members such that the radius of gyration of a cross member and the half panel width on either side is not less than [0.02] times the lesser of its height or length.
- The effective thickness is the total thickness of a single leaf panel or the equivalent thickness of a two-leaf panel in respect of compressive stability.

(5) Holes may be disregarded only when they do not significantly impair the

action of the diagonal bracing member described in L2(4).

a. Such holes are any located immediately adjacent to the surrounding frame and within the middle third of the infill adjacent to the beams or columns. The maximum dimension of any hole in the panel perpendicular to the adjacent frame member must not exceed one tenth of either the height or the length of the panel, whichever is the smaller.

b. Larger holes may be permitted such that they do not exceed 40% of the panel length nor 40% of the panel height, providing they are restricted to the middle strip in one direction and providing the holes are trimmed by cross members sufficient to carry the local out-of-plane loading and have a tensile and flexural strength and stiffness not less than those of the belts specified in clause 9.5.3 of prENV 1998-1. These may be of reinforced concrete or steel (for example lintel beams and wind posts). To improve stability under out-of-plane loading these cross members should be connected to the frame.

(6) The longitudinal axes of the framing members should lie within the middle third of the width of the panel. For panels of cavity construction these axes should lie within 40mm of the centroid of the forces in the two leaves.

(7) Transverse loading on a panel may be assumed to act independently of in-plane loading.

(8) Care should be taken to avoid gaps between the infill and the frame except as discussed in L6. The use of gaps to reduce damage to the panels is beyond the scope of these clauses.

(9) Ties between the panel and the frame are not necessary to develop the in-plane shear resistance of infilled frames, but they are required to resist out-of-plane effects. Where ties are sufficient to prevent separation at the ultimate limit state, reinforcement should be provided to carry the in-plane tensile stresses induced in the panel. Tensile stresses induced by wire ties embedded in the masonry and fixed to the inside of the frame may generally be neglected. Suitable ties include the reinforcement in reinforced renders and the cross members in (4) and (5) when connected to the frame.

(10) The effect of shear connectors and ties on in-plane action is considered in Clause L5.

(11) Where K as defined in L2 exceeds:

- [12] or,
- for poorly fitting panels (those requiring stiffness correction in L6(1)) or other panels with some gaps [9].

(12) For concrete infills complying prEN 1998-1 and monolithic with the frame the resistance may be conservatively assessed by the procedures in this annex.

L.2 Procedure for Assessing the Resistance of Framed Panels Subject to Horizontal Loading

The applied horizontal force H at the ultimate limit state shall not exceed:

- For masonry panels the least of the shear strength $H_{rd,v}$, the indirect tensile strength $H_{rd,t}$ and the compressive strength $H_{rd,c}$, except that no check is necessary on the indirect tensile strength for grades (ii) and (iii) mortar.
- For concrete infills, either unreinforced or nominally reinforced, monolithic with or debonded from the frame, the lesser of the indirect tensile strength $H_{rd,t}$ and the compressive strength $H_{rd,c}$, where:

i) For masonry infills (shear failure on bedding planes)

$$H_{rd,v} = \frac{f_{bs} L t k_1}{1.43\gamma_{mv} - \mu[0.8(h/L) - 0.20]} \quad \text{(L1)}$$

ii) For concrete infills and for solid masonry infills with Grade 1 mortar (diagonal tension failure)

$$H_{rd,t} = \frac{Lt(f_k/\gamma_m)}{5.8} \quad \text{(L2)}$$

iii) For all infills (crushing of masonry)

$$H_{rd,c} = 1.7\,(f_k/\gamma_m)\,h\,t\,k_2\cos^2\theta/K \quad \text{(L3)}$$

where

L is the length of the infill panel

h is the height of the infill panel

t is the smaller of the second moments of area of the column sections about their axes perpendicular to the plane of the frame

θ is the slope of the diagonal of the infill panel to the horizontal

μ is the coefficient of friction for movement on the horizontal mortar joints of the infill panel, calculated in accordance with Eurocode 6

f_{bs} is the characteristic bond shear strength of the brickwork in accordance with Eurocode 6

f_k is the characteristic compressive strength of the masonry in the horizontal direction in accordance with Eurocode 6, except where the strength vertically is less than 65% of the strength horizontally the strength horizontally shall be reduced by the factor (the ratio of the vertical to horizontal strength)$^{0.33}$

γ_{mv} is 2/3 of the partial safety factor for the shear strength of the panel in accordance with Eurocode 6, (there termed γ_m) but not less than 1.5

γ_m is 2/3 of the partial safety factor for the compressive strength of the panel at the ultimate limit state in accordance with Eurocode 6, but not less than 2.0

E is the stiffness (Young's modulus) of the columns

E_w is stiffness of the masonry in direct compression

k_1 a reduction coefficient: 0.80 for central windows or other holes satisfying L1(5)b

k_2 a reduction coefficient of:
0.75 for pinned or semi-rigid frame connections;
0.67 for central windows or other holes satisfying L1(5)b;
0.33 for pinned or semi-rigid connections and with central windows or holes satisfying L1(5)b.

K is $(E_w t h^3/EI_c)^{1/4}$, but not less than 3.5

(2) Where the beams are less stiff than the columns, the stiffness of the columns may conservatively be taken as that of the more flexible beam.

(3) γ_m values for the materials in earthquake resistant design/ assessment take into account deterioration of the infill under repeated

loading. The minimum values specified are intended as a deterrent to panel failure.

(4) In the global analysis the panel may be conservatively represented by a diagonal bracing member in compression, of elastic modulus E_w and area A_w as follows:

$A_w = \alpha k_2 L t \sec\theta$ where

α may be taken as 0.10 or the value given by 0.26 $K^{-0.4}$ if higher,

and K is as defined earlier in this sub-clause

E_w shall not be taken greater than 7kN/mm².

Where there are holes in panels the stiffness of panels may need to be adjusted. In the absence of other guidance the correction may be taken as k_2 .

(5) The resistance of the panel to in-plane horizontal loading need not be taken as less than that of confined masonry in prEN1998-1.

L.3 Procedure for the Assessment of the Frame

(1) Beams and columns should be designed for the worst combination of dead, imposed and horizontal actions. The axial forces in the members caused by the horizontal loading in the plane of the panels may be determined by the static analysis of an equivalent frame with pin-jointed or rigid-jointed connections, in which panels are represented as diagonal pin-jointed bracing struts by the value of A_w given in L2(4). The panels should be assumed to carry no gravity loads besides their own self weight. Any restraint to a beam or column from an adjacent panel should be neglected. Stress resultants from horizontal actions in panels without holes or with holes complying with L1(5)a may be taken as:

a. A moment over the entire length of each column of Hh/20, applied in the plane of the frame, disregarding the moments in (d) and (e) below;

b. A shear force in each column within a length of L/10 from each end, of γ_{Rd} H, where:

H is the larger value of the horizontal component of the forces in adjacent panels;

$\gamma_{Rd} = 1 + 0.25(H_{Rd}/H - 1)$;

H_{Rd} is the lesser//least of expressions L1 to L3 as appropriate, where γ_m and γ_{mv} are taken as 1.0

c. In infilled bays with rigid or monolithic joints a shear force of $2M_r/h$ carried over the full length of each column, where M_r is the moment of resistance of the column taking into account the axial load.

d. When the beam below a framed panel is not restrained by a panel below, a mid-span sagging moment in the beam of Hh/20.

e. When the beam above a frame panel is not restrained by a panel above, a midspan hogging moment in the beam of Hh/20.

f. A shear force of Hh/L in the beam above a panel, within a length of L/10 from each end in solid panels, which may not be designed as a shear hinge.

g. A shear force of Hh/L in the beam below a panel, within a length of L/10 from each end, which may not be designed as a shear hinge.

(2) Where there are holes complying with L1(5)b the moments in L3(1)a, d and e above and the lengths subject to the shears should be increased by a factor of 2.5.

(3) In semi-rigid construction M_r in (1)c should be replaced by the greater column end moment.

L.4 Additional Requirements for Reinforced Concrete Frames

(1) Unless calculations are performed to justify other assumptions the second moment of area of the frame members should be based on the cracked section properties. The uncracked section however may be used in analyses for checking the condition of cracking in the panels if E_w is not less than 7kN/mm² (Equations L1 and L2).

(2) The shear resistance of the columns in monolithic construction shall be checked for two conditions as follows:

i) The shear of 2 Mr/h in L3(1)c above should be considered over a shear span of h/2.

ii) For the check in L3(1)b, the variable strut inclination method of prENV 1992-1, may be used.

iii) Where the analysis shows the frame members are subject to direct tension the shear strength of the reinforced concrete section shall be appropriately reduced.

L.5 Shear Connectors and Internal Posts

(1) Shear connectors provided to resist out-of-plane forces may be designed disregarding in-plane effects, except that ties should be provided through the panel capable of carrying 0.10H, where H is defined in L2.

(2) If shear connectors are provided across horizontal boundaries between the frame and the infill (as might be sometimes required to reduce the column shear) these should be designed to resist the full force H.

L.6 Gaps Between Panel and Frame

(1) Gaps between the panel and the frame shall be taken into account in the analysis if the sum of the gaps is greater than [1.5]mm. Where the sum of the gaps is less than [2.5]mm, this may be taken into account by reducing the stiffness of the diagonal to represent the closure of the gap(s) under the force H_{Rd} in L3(1)b. Where there are gaps between [1.5]mm and [2.5]mm, the provisions of L1(11) apply.

(2) Particular attention must be paid to restrain the panel from out-of-plane failure where there is a gap, and, for vertical gaps, at both ends of the panel.

NOTABLE EARTHQUAKES SEPTEMBER 2002 - NOVEMBER 2002

Reported by British Geological Survey

YEAR	DAY	MON	TIME UTC	LAT	LON	DEP KM	MAGNITUDES ML	MW	LOCATION
2002	**06**	**SEP**	**01:21**	**38.38N**	**13.70E**	**5**		**5.9**	**SICILY, ITALY**
			Two people killed, 20 injured and several buildings damaged in the Palermo area.						
2002	**08**	**SEP**	**18:44**	**3.30S**	**142.95E**	**13**		**7.6**	**PAPUA NEW GUINEA**
			Four people killed, at least 70 injured, over 700 buildings destroyed or damaged and several water tanks, pipelines and bridges damaged in the Wewak area. Damage also caused by local tsunami and landslides.						
2002	**13**	**SEP**	**22:28**	**13.04N**	**93.07E**	**21**		**6.5**	**ANDAMAN ISLANDS**
			Two people killed and over 40 houses destroyed on Middle Andaman. A local tsunami caused damage to several buildings at Ariel Bay.						
2002	**20**	**SEP**	**15:43**	**1.68S**	**134.23E**	**10**		**6.4**	**IRIAN JAYA REGION**
			At least 31 houses damaged at Ransiki.						
2002	**22**	**SEP**	**23:53**	**52.52N**	**2.15W**	**9**	**4.8**		**DUDLEY**
			One person injured at Mansfield. At least one chimney collapsed and other minor damage reported in the Dudley area. Felt from Liverpool to London and from Lincolnshire to Wales. Maximum Intensity of at least 6 EMS.						
2002	**23**	**SEP**	**03:32**	**52.52N**	**2.14W**	**9**	**2.7**		**DUDLEY**
			Felt in the West Midlands with intensities of at least 3 EMS.						
2002	**03**	**OCT**	**16:08**	**23.32N**	**108.53W**	**10**		**6.5**	**GULF OF CALIFORNIA**
			Felt strongly throughout the epicentral area especially at Mazatlan.						
2002	**09**	**OCT**	**21:03**	**55.12N**	**3.61W**	**7**	**1.3**		**DUMFRIES**
			Felt in the Tinwald area with intensities of 3 EMS.						
2002	**10**	**OCT**	**10:50**	**1.76S**	**134.29E**	**10**		**7.6**	**IRIAN JAYA REGION**
			Eight people killed, at least 632 injured, more than 1,900 buildings destroyed or damaged in the Ransiki-Manokwari area. A surface fault 3 km long occurred at Ransiki. Many houses were flooded by a local tsunami with wave heights of 3-5 metres.						
2002	**10**	**OCT**	**12:28**	**1.71S**	**133.97E**	**10**		**6.7**	**IRIAN JAYA REGION**
			Further damage occurred in the Ransiki-Manokwari area.						
2002	**12**	**OCT**	**20:09**	**8.30S**	**71.74W**	**534**		**6.9**	**WESTERN BRAZIL**
			Felt throughout Brazil and at Pucallpa, Peru.						
2002	**21**	**OCT**	**11:43.34**	**53.48N**	**2.20W**	**5**	**3.9**		**GTR MANCHESTER**
2002	**21**	**OCT**	**11:43.56**	**53.48N**	**2.21W**	**5**	**3.5**		**GTR MANCHESTER**
			The largest 2 events in a swarm of over 107 earthquakes, detected on the BGS seismic network, to occur in the Manchester area. The first event in the swarm occurred on the 19 October and continued throughout the rest of October and November. The magnitudes of the events ranged between 1.3 and 3.9 ML and over 35 of them were reported felt, to BGS, with intensities between 2 and 5 EMS.						
2002	**23**	**OCT**	**11:27**	**63.58N**	**148.09W**	**14**		**6.7**	**CENTRAL ALASKA**
			Felt throughout the epicentral region and items were knocked from shelves in Anchorage.						
2002	**24**	**OCT**	**06:08**	**1.90S**	**28.90E**	**11**		**6.2**	**REPUBLIC OF CONGO**
			Two people killed at Goma. Damage reported from Goma, Lwiro and Mugeri in the Democratic Republic of Congo and also from Kigali, Rwanda.						
2002	**29**	**OCT**	**10:02**	**37.52N**	**15.13E**	**10**		**4.3**	**SICILY, ITALY**
			Nine people injured and many homes, shops and churches damaged in the Santa Venerino area.						
2002	**31**	**OCT**	**10:32**	**41.78N**	**14.91E**	**10**		**5.9**	**SOUTHERN ITALY**
			At least 29 people killed, many injured, a number missing and extensive damage in the Campobasso area.						
2002	**01**	**NOV**	**15:09**	**41.78N**	**14.87E**	**10**		**5.8**	**SOUTHERN ITALY**
			Three people injured and additional damage at San Giuliano di Puglia.						
2002	**01**	**NOV**	**22:09**	**35.58N**	**74.70E**	**33**		**5.4**	**NW KASHMIR**
			At least 17 people killed, 65 injured and extensive damage reported in northern Kashmir. Landslides also reported from the epicentral area.						
2002	**02**	**NOV**	**01:26**	**2.99N**	**96.08E**	**49**		**7.4**	**NORTHERN SUMATRA**
			Three people killed, 60 injured and extensive damage on Simeulue Island.						
2002	**03**	**NOV**	**22:12**	**63.52N**	**147.53W**	**5**		**7.9**	**CENTRAL ALASKA**
			One person injured and extensive damage caused to roads and highways. Items knocked from shelves in Denali National Park, Glenallen and Tok. Some supports on the trans-Atlantic pipeline were damaged and operations were suspended. Damage estimated at US$20million.						
2002	**07**	**NOV**	**15:14**	**51.17N**	**179.46E**	**33**		**6.6**	**ALEUTIAN ISLANDS**
2002	**15**	**NOV**	**19:58**	**55.93S**	**35.90W**	**33**		**6.7**	**STH GEORGIA ISLAND**
2002	**17**	**NOV**	**04:53**	**47.98N**	**146.28E**	**499**		**7.3**	**NW KURIL ISLANDS**
2002	**19**	**NOV**	**21:15**	**49.19N**	**2.08W**	**13**	**2.5**		**JERSEY**
			Felt on Jersey with intensities of at least 3 EMS.						
2002	**20**	**NOV**	**21:32**	**35.39N**	**74.55E**	**33**		**6.4**	**NW KASHMIR**
			At least 30 people killed, many injured and extensive damage in the Astor Valley and Dashkin areas. More than 15,000 people left homeless in the region.						

Issued by: Davie Galloway, British Geological Survey, December 2002.

"Special Earthquake Session at the Fifth International Conference on Case Histories in Geotechnical Engineering". Call for Papers.

The conference is to be held in New York, NY, USA on April 13-17, 2004 and papers are invited on the following topics:

A. *Geotechnical Aspects of Italy 2002 and Alaska 2002 Earthquakes* including:
- Liquefaction;
- Ground motion and amplification - comparison with other recent earthquakes;
- Failure of ground;
- Damage to geotechnical structures;
- Similar topics.

B. *Mitigation and Design for Liquefaction* including:
- Analysis and design of laterally loaded large diameter piles and pile groups using advanced calibrated computer models in as-is and liquefied conditions;
- Earthquake drains for mitigation of liquefaction;
- Load tests for piles for liquefied and improved soil ground conditions;
- Lateral load testing of pile groups;
- Lateral load tests on large diameter piles;
- Calibration studies of the DSSI problem from recent earthquakes in seisomlogical-geotechnical-structural hand shake in the performance based design conditions of ground deformations related to soil liquefaction;
- And similar topics.

The web site is www.umr.edu/~eqconf/5thCHConf. If you are interested in contributing to this Special Session or if you have any questions please let the conference organisers know. You can reach them, Shamsher Prakash, Conference Chairman, by email at prakash@umr.edu or email Wanda Furniss, Conference Secretary, at furnissw@umr.edu. The last date for abstracts is April 1, 2003 and the last date for full manuscripts is August 1, 2003.

Online Report

The Health and Safety Executive have recently placed a publication related to offshore earthquake hazard assessment on their web site at www.hse.gov.uk

ISET Trifunac Award

The Indian Society of Earthquake Technology are currently inviting nominations for the "ISET Trifunac Award for Significant Contributions in Strong Motion Earthquake Studies".

Nominations are required by 15 February 2003. For more details contact Dr.Vinay K. Gupta at vinaykg@iitk.ac.in or see the SECED website.

Forthcoming Events

29 January 2003
The Earth as a Musical Instrument - Frank Scherbaum.
ICE 5.30pm

26 February 2003
Seismic Walkdown - a Technique for Evaluating Seismic Capability - Tim Allmark and Colin Hughes.
ICE 5.30pm

26 March 2003
Polymer Reinforcement - Professor Sofronie.
ICE 5.30pm

30 April 2003
This Year's Earthquake

28 May 2003
9th Mallet-Milne Lecture: M.J.N Priestley
"*Revisiting Myths and Fallacies in Earthquake Engineering*"

SECED Newsletter

The SECED Newsletter is published quarterly. Contributions are welcome and manuscripts should be sent on a PC compatible disk or directly by Email. Copy typed on one side of the paper only is also acceptable.

Diagrams should be sharply defined and prepared in a form suitable for direct reproduction. Photographs should be high quality (black and white prints are preferred). Diagrams and photographs are only returned to the authors on request. Diagrams and pictures may also be sent by Email (GIF format is preferred).

Articles should be sent to:

John Sawyer,
Editor SECED Newsletter,
Scott Wilson,
Scott House,
Basingstoke,
Hants,
RG21 4JG,
UK.

Email: john.sawyer@scottwilson.com

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 Geophysical 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,
Great George Street,
London SW1P 3AA, UK.

SECED Website

Visit the SECED website which can be found at http://www.seced.org.uk for additional information and links to items that will be of interest to SECED members.
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