Rock failure in collapse and caprock sinkholes
Rock failure under imposed load over caves

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Rock failure in collapse and caprock sinkholes

3.1 KARSTIC COLLAPSE

All ground voids constitute elements of weakness within a rock mass, and karst is distinguished by having the largest natural voids, where roof failure can create a significant geohazard. The natural consequence of progressive roof failure is upward void migration, which may reach the surface where it causes instantaneous major subsidence in the form of a collapse sinkhole. Where the roof failure migrates up through non-karstic rocks, the surface failure in an outcrop of insoluble rock creates a caprock sinkhole. Both these forms of sinkhole involve failure and collapse of bedrock, and are therefore distinct from the subsidence sinkholes where soil cover is flushed into stable rock fissures (Chapter 4).

Collapse and caprock sinkholes are initiated where cave passages or cave chambers are enlarged beyond the limits of their own roof rock stability. Roof collapse is a natural and automatic process in all karstic caves, though its development to a scale that influences surface stability is dependent on the necessary geological structures and the long periods of geological time for processes to mature. Failure of a cave roof can be initiated or accelerated by imposed loads from construction works, and is therefore a major geohazard where large caves exist at shallow depths (Chapter 7).

Processes of cave roof collapse within gypsum and salt are comparable to those in limestone, but are distinguished by the much lower mechanical rock strengths. Collapse sinkholes are formed in both rocks, and caprock sinkholes are proportionately more important than they are in limestone. Breccia pipes are related structures that originate with deep-seated dissolution in these rocks, but are rare over limestone. Collapse sinkholes also develop over lava caves, but the origins, structures and formative processes of both the caves and the sinkholes are very different from those of karst landforms, as they are unrelated to post-genetic dissolution (Chapter 6).

3.2 COLLAPSE OF CAVE CHAMBERS

The stability of a limestone cave is a function of its unsupported span and the structural integrity of its roof rock. Most caves are structurally sound; relationships between cave widths and the rock mass quality of cavernous limestones suggests that the majority of caves would require little or no support if they were regarded as engineered structures (Waltham and Fookes, 2003). A tubular tunnel dissolved out of massive limestone far beneath the surface is extremely stable. In contrast, even the smallest caves can collapse in zones of heavily fractured rock or beneath very thin roof slabs (Figure 3.1).

Cave passages are typically less than 10 m wide in most temperate regions, though chambers formed at optimum sites or by passage coalescence are common to about 50 m across, and isolated larger caverns do exist. In tropical regions, caves 30 m across are not unusual, and numerous chambers (often known as caverns) are over 100 m wide. Surface collapse is related to cave size, and there is therefore a climatic influence on the size and frequency of collapse sinkholes in limestone karst. (Parameters for caves in gypsum and salt are different, and are discussed below.)

The positions, shapes and sizes of both cave passages and cave chambers are guided by the structural and stratigraphic features of the host rock. The geological influences can be recognised in nearly all accessible caves, but the multiple choices offered by complex rock structures mean that the positions of neither caves nor enlarged chambers can be predicted ahead of exploration. Large caverns tend to

Figure 3.1. Collapse of the thin limestone roof over a small cave in the side of a wadi in Jordan; the two beds that roofed the cave, and partially survive in the entrance arch, are probably an indurated duricrust over a more easily eroded blocky limestone.
TW.

Figure 3.2. The world's largest known cavern – Sarawak Chamber, in the Mulu karst on Borneo. Eight cavers with very large flashguns just light the multiple arches that form the roof, with the far wall 300 m from the camera.
Photo: Jerry Wooldridge.

form where a weak, thinly bedded or densely fractured limestone is underlain by a stronger or more massive unit, so that rapid erosion can progress beneath a stable roof (Gilli, 1986). Favourable sites for cave chambers occur where larger flows of water enter the karst, from rivers off insoluble rocks or where flows are concentrated through breaches in shale beds within the limestone sequence. Many chambers have formed by coalescence of adjacent passages and shafts with intervening wall collapse, by lateral wall undercutting, or by enhanced mixing-corrosion (Bögli, 1964) at the junctions of major passages. The world's largest known cavern, 700 m long and 300 m wide in the Mulu karst of Sarawak (Figure 3.2), is formed in massive limestone with bedding planes 15–20 m apart, where a major cave river cut laterally on a favourable geological structure (Waltham, 1997). In contrast, the famous Carlsbad Cavern, in New Mexico, was cut in almost structureless reef limestone by slow-moving water enriched in sulphuric acid that was derived from hydrocarbons in adjacent basins.

3.2.1 Cave roof breakdown by bed failure

Roof collapse by progressive roof failure is widespread in limestone caves, and can be very conspicuous in any zones of more thinly bedded limestones (Figure 3.3). These contrast with roof profiles that retain their original dissolutional features in massive limestones or within single very thick beds, and can span large voids with no sign of breakdown (Figure 3.4). The simplest analysis of a cave roof in bedded limestone is to treat the beds as beams failing under their own weight between the cave walls.

For an unsupported beam of span length L, of unit width, of depth d, of unit weight γ and of weight W $(= Ld\gamma)$, the bending moment $M = WL/12 = L^2d\gamma/12$, and the section modulus $Z = d^2/6$.

Assuming elastic behaviour of the rock, failure occurs when the tensile strength T is reached at M/Z in the outer surface of the deforming beam.

Consequently the stable beam depth (or bed thickness) $d = L^2\gamma/2T$.

This assumes that the beam is constrained at its ends, which is the normal situation in a rock mass. If the beam is unconstrained, due to the presence of open fissures, the bending moment is $WL/8$ and the stable beam depth $d = 6L^2\gamma/8T$.

For limestone, the unit weight γ may be taken as $26\,\mathrm{kN/m^3}$.

The critical factor is therefore the tensile strength T of the limestone, and, for a typical cavernous limestone of unconfined compressive strength around 100 MPa, this may be taken as about 5 MPa. The stability envelope for constrained limestone roof beams of this tensile strength may therefore be determined (Figure 3.5).

This representation of cave roof stability by beam integrity is only a simplification of reality, though it has been used to design safe working under bedded rocks in

Figure 3.3. Cave passages with flat limestone roofs developed by progressive failures along bedding planes; on the left in Mammoth Cave, Kentucky, and on the right in Ogof Agen Allwedd, U.K.; in both caves the roof is some metres above the original dissolutional passage. TW.

Figure 3.4. Massive limestone forming the roof of the Niah Great Cave, Sarawak, with no signs of breakdown to destroy its dissolutional sculpture.
TW.

British iron ore mines (Whittaker and Reddish, 1989). Factors that reduce stability include rock jointing that weakens or destroys the beam, any overburden load that bears on the beam, dipping geological structures that complicate the roof profile and time that allows strength decrease from the short-term levels. Factors that increase stability include cohesion within the shale that occupies most bedding planes in unweathered limestone, irregular fractures that allow blocks to lock against each other and any development of compression arches through multiple bed sequences.

Data on approximate bed thicknesses and roof spans, from numerous caves around the world that have either failed or remain stable are included in Figure 3.5. The observed data show a reasonable correlation with the calculated beam envelope, although a rather better limit is given by the simple relationship of $L = 17d$. Earlier analyses of roof beam failure (Davies, 1951; White and White, 1969; White, 1988) used shear strength in place of tensile strength for failure and therefore achieved an unrealistically large stability envelope (Figure 3.5). Rock strength decreases with time due to fracture propagation from stress points at grain boundaries on critical surfaces, and it is realistic to cite a long-term decrease of tensile strength to about 30% of the short-term value (Tharp, 1995). However, the stability envelope based on this reduced tensile strength is small in relation to observed data (Figure 3.5). Unconstrained displacement of the rock beam ends also reduces the stability envelope, and appears to be unrealistic as it has even less correlation with observed data. Analysis of mine roofs in strong bedded limestone was correlated with monitoring data from an experimental mine room (Merrill, 1957) to define a stability envelope very similar to that proposed above for cave roofs (Figure 3.5).

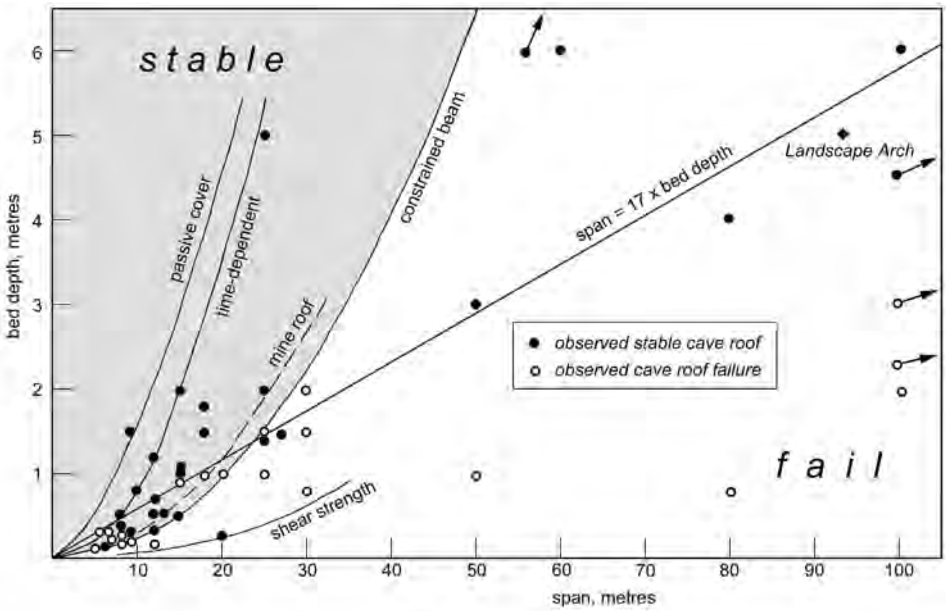

Figure 3.5. Correlation between bed thickness and chamber span width with respect to the failure of intact rock beams over wide caves. The shaded stability envelope is calculated for constrained beams of rock with a tensile strength of 5,000 kPa. The other envelopes are (from the left) calculated for a passive cover as thick as the cave width, based on long-term tensile failure of limestone at 30% of its short-term strength (Tharp, 1995), determined for roof failure in limestone mines (Merrill, 1957), the simplistic relationship of $L = 17d$ (span width $= 17 \times$ bed depth) and calculated for beam failure by employing its shear strength (White, 1988). Point data refer to failed and/or stable observed roofs in 43 caves and also the sandstone Landscape Arch, Utah.

Where a cave roof is reliant on support by a single thick bed of rock, any cover of more thinly-bedded rock and/or unconsolidated soil adds a distributed load to the roof beam, and thereby reduces the stability envelope (Figure 3.5). The lack of correlation between this envelope and the observed data suggests that this is not the general case. Most roof rock masses appear to be internally supportive, with each bed or element providing its own support.

Cave roof failures that lie inside the stability envelope (Figure 3.5) may be ascribed to rock fractures that destroy beam integrity. Cave chambers only survive in rock with relatively intact beds. Heavily fractured rock does not appear in Figure 3.5 because chamber roofs collapse before they can become accessible and observable, and erosion is directed to create tall and narrow fissure caves instead of chambers. Stable caves that lie outside the stability envelope appear to gain their support from compression arches, and not purely from rock beams.

3.2.2 Stable arch development in cave roofs

Distortion of gravitational stress around a ground cavity creates an arched compression zone over the roof and into the walls, with a tension zone in the roof immediately beneath the arch. Most natural cave passages and chambers evolve towards an arched profile as rock falls away from the tensile zone where it is immaterial to the total roof stability. The large entrance chamber of Tham En (Figure 3.6), in the karst of Laos, has evolved to a stable arched roof profile in massive rock after stoping up through thinly bedded limestone (Waltham and Middleton, 2000). A cantilever is weaker than a beam, but there are zones within the compression arch where beds remain in cantilever and contribute to support of a wider span. Within almost any arch there are zones where bedding planes and fractures are oblique to the compressive stress and the underside is unconfined. Shear failure in these zones leads to development of higher arches to achieve stability. Uncontrolled breakdown of mine roofs generally leads to the development of a stable arch profile when the arch rise equals about half the span width (Franklin, 1989), a profile that is close to that of the theoretical tension zone above a cavity. Even in well-bedded limestones, most cave chambers are observed to have arched roof profiles, and most of these have a span much greater than the rise of the arch; the Tham En roof arch rises by less than a third of its span.

Natural cavern roofs that have evolved slowly and only carry their self-load are stable in very low arch profiles in the strong limestones that can contain large caves. For a given roof thickness and span, there is a range of possible arches that balance arch height against arch thickness. A higher parabolic profile develops in weak rocks, notably mine roofs in thinly bedded coal measures, but strong limestones are stable in much lower arches, especially those that are laterally constrained by high horizontal stresses in the wall rocks; furthermore, the thick roof over a deeply buried cave chamber has space to develop an arch of almost any profile. In practice,

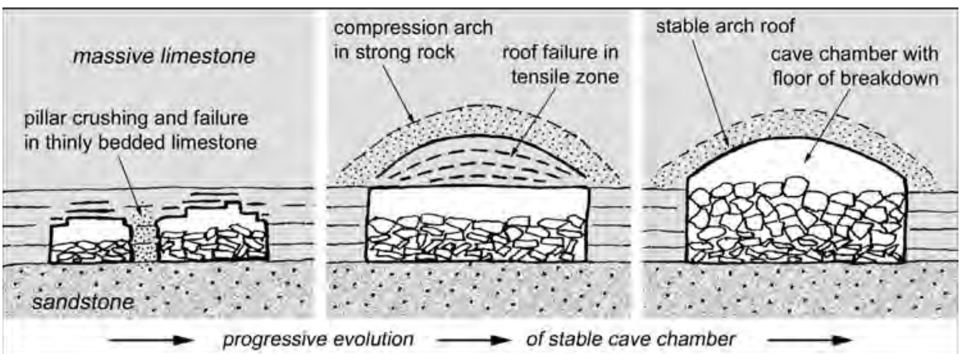

Figure 3.6. Development of a compression arch in massive limestone over the southern entrance chamber of Tham En, Laos; roof failure and passage enlargement in thinly bedded limestones was followed by evolution of an arched cave roof by stoping of rock from the tension zone.

Figure 3.7. The stable, low-profile, compression arch that forms Landscape Arch in the sandstone of Arches National Park, Utah, spanning 93 m and only 5 m thick.
TW.

the arch profile is determined by the details of its geological structure, and these remain unseen in an intact arch. Analyses of arched cave roofs are therefore very approximate, as they have to rely on estimates of the reduction of rock mass strength due to the unseen fractures. These become critical to ground stability where the rock may be stressed by imposed construction loads (see Chapter 7).

A compression arch is much stronger than a beam in the same material, and natural arches can be very thin. The spectacularly thin Landscape Arch (Figure 3.7), in Utah, demonstrates how strong rock is under compression. It is weathered out of sandstone that is weaker than most cavernous limestones, and is not devoid of fractures. Regarded as one bed, it plots on Figure 3.5 very close to the stability limit of $L = 17d$. An arched cave roof developed by phreatic dissolution (below the water table) creates a naturally stable profile, but is rarely wider than 20 m; most large cavern roofs have arch profiles that have evolved by roof stoping within the tensile zone. A roof arch in a limestone anticline could gain strength where its blocks of rock between radiating joints perpendicular to the bedding mimic a stable voussoir arch; this is very rare in nature, but may contribute to the stability of the enormous Sarawak Chamber in the Mulu karst (Gilli, 1993).

3.2.3 Breakdown processes

Cave roof breakdown is generally initiated by lateral stream undercutting that is excessive for the roof span strength. This becomes self-propagating where a stream is then diverted into an undercut around a pile of fallen breakdown. Roof collapse may also be initiated where a phreatic chamber (formed below the water table) is first drained by rejuvenation. Buoyant support in water takes nearly 40% of the rock load, but this is imposed immediately on drainage, when it probably creates the largest short-term stress increase in the lifetime of many chambers. Hydrostatic pressure due to the aquifer head is not relevant as it is applied equally as joint water pressure within roof fissures. Mineral wedging (notably sulphate growth) can contribute to roof breakdown in some environments (White and White, 2003), while frost action becomes significant in caverns at shallow depths, including entrance chambers, outside the tropical regions.

While the final collapse of a cavern roof is an instantaneous event, progressive stoping and cavity migration may extend over geological timescales, and there is almost no available data that records roof evolution. Roof stoping in abandoned mines has been known to migrate through tens of metres of cover to cause surface crown holes (also known as chimney subsidences) within only months of the loss of support in the underlying mine. Surface failures have occurred within 3–10 years of the initial roof collapse in ironstone mines at depths of 100 m in Britain (Whittaker and Reddish, 1989). This rapid stoping only occurred where water entered from overlying aquifers within sequences of weak rocks, so the analogy to cavernous limestone is limited. In contrast, some crown holes have developed hundreds of years after mine abandonment, but these data apply mainly to weak sedimentary rocks of the Coal Measures. In Quebec, Canada, roof failure of a mined cavity at a depth of 70 m reached the surface by uncontrolled progressive stoping within less than two days (Franklin, 1989). Major surface collapses over large brining cavities in salt have followed stoping failure through hundreds of metres of rock within only months or years (Section 3.4.3), but these are also failures in weak cover rocks.

In the Pokhara basin of Nepal, natural cave excavation and subsequent total collapse of a roof about 20 m thick appears to have occurred all within about 500 years (which is the likely age of the host limestone), but this is an exceptional site of powerful erosion in weak rock (Waltham, 1996). Most cave chambers can only be dated as many thousands of years old, and early roof failures are likely to be followed by more stable conditions as arched profiles evolve. Cave roof stoping rates remain unknown, but are likely to be much lower in strong limestones than in the situations cited above. Roof reduction by progressive stoping failure presents a karst geohazard that is negligible when compared to rock failure accelerated by imposed loads from construction works over an unknown cavern.

There is limited available data on cavern roof collapses in gypsum and salt, both of which are mechanically weaker than most limestones. Large chambers in salt are created by brining operations, and, unless carefully controlled, many do eventually collapse to form breccia pipes and/or sinkholes (see below). Cave chambers wider than about 25 m are almost unknown in gypsum. Roof collapse is complicated by the material's plastic deformation and its transitions to and from anhydrite, so that beds can bend and curve away from cave roofs without breaking. Dissolutional undercutting and fissure opening is so rapid in gypsum that cave streams can promote collapse of the weak roof rock within only years or decades of flow past an originally stable site. This is especially important where streams are diverted by piles of collapse debris so that they undercut chamber walls in gypsum and cause a renewal or widening of the collapse within only tens of years.

3.3 COLLAPSE SINKHOLES

Where cave roof failures propagate through to an exposed karst surface, they become collapse sinkholes. These appear as surface depressions from 1 m to 300 m across, typically with some rock outcrops, scars or walls in their perimeter remaining from the collapse processes that modified the initial dissolutional cavity or cavities within the bedrock. Collapse sinkholes have very variable depth/width ratios, commonly >1, thereby distinguishing them from most solution sinkholes. They may or may not have a soil cover, but their key factor is the collapse of bedrock and not only of the soil, as in some subsidence sinkholes.

3.3.1 Collapse sinkholes in limestones

The simplest type of collapse sinkhole is created by the total failure of the roof of a shallow cave. A young collapse sinkhole has almost vertical rock walls and a debris floor sloping down into an open cave passage (Figure 3.8). Over time this degrades, so that an old collapse sinkhole has flared and broken walls, is likely to have accumulated a wind-blown or in-washed soil on its floor and probably has no open entrance into the cave below (Jennings, 1975). Further degradation leaves it inactive (Figure 3.8), with an alluviated floor, a rounded plan-shape and only traces of rock wall that distinguish it from a solution sinkhole. In contrast, an active collapse sinkhole is likely to have a jagged outline that reflects the original cave shape, and it may have almost no debris on its floor, where the fallen roof breakdown has been removed by the stream or river that flows across it. The Slovenian karst at Skocjan provides very fine but very large examples of collapse sinkholes (Box 3.1), and their morphological features could be reduced by scale factors of 5 or 50 to match numerous other collapsed sinkholes in karst regions around the world. Cenotes and many blue holes are cliff-ringed lakes that are collapsed sinkholes into flooded caves (Figure 3.9); some lakes are floored by debris piles which block access to the outlets, but many cenotes in Mexico's Yucatan karst are windows into networks of flooded caves that extend for many kilometres (Beddows, 2004). As open windows into highly productive aquifers, these cenotes can constitute valuable water resources; the Otjikoto cenote lake

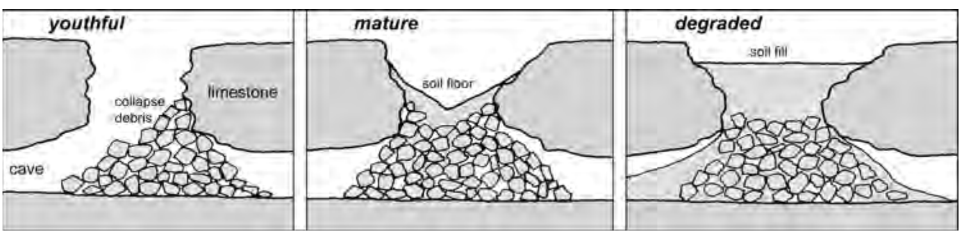

Figure 3.8. Three profiles that demonstrate the evolution of collapse sinkholes from youthful to mature to degraded.
Partly after Jennings (1975).

Figure 3.9. The splendid Otjikoto cenote in the Otavo karst of Namibia; the water is 120 m deep, but there is no known access to the flooded caves that extend either way from the cenote floor.
TW.

(Figure 3.9), in Namibia, has been pumped for many years without any decline of its level.

While collapse sinkholes may form by the failure of individual cave chambers, many are formed by multiple and progressive collapses over a zone of shafts, caverns and passages. Cave passage and shaft widening can cause adjacent features to coalesce as intervening walls of rock are thinned until they collapse and take with them the parts of the roof that they supported. In this manner large collapse sinkholes can be created in areas of heavily fractured and fissured rock where the roof of a single large chamber could not have survived even temporarily. Major collapse features are more common than large chambers inside cave systems, and this also accounts for the numbers of significant collapse sinkholes along the outcrops of faults. In the Slovenian karst, Rakovska kukava is a collapse sinkhole 240 m across and 70 m deep with steep rock sides. It is more than 15 times larger than the largest known cave chamber in the area, in limestones so fractured that larger underground voids are inconceivable, and the sinkhole is considered to have formed gradually and progressively by serial collapses into an underlying cave. As it is believed to have developed from a small hole that has grown steadily larger, it has been called a tumour doline (or sinkhole) that is a variety of collapse sinkhole where no large cave chamber ever existed (Šušteršič, 1998). Tumour sinkholes fall within the spectrum of features transitional between collapse and solution sinkholes, where dissolutional enlargement of numerous fissures and small caves is followed by repeated small-scale collapses.

BOX 3.1. COLLAPSE SINKHOLE

An example – Skocjanske Jama, Slovenia

The river cave of Skocjanske Jama is cut into Cretaceous limestones on the south-east side of the Kras plateau in southern Slovenia. The River Reka sinks into the cave's massive entrance, and beyond 200 m of passage it flows across the floors of two very large collapse sinkholes (Figure 3.1.1). Mala (*Small*) Sinkhole is 130 m in diameter, while Velika (*Great*) Sinkhole is more than 150 m by 250 m in plan (Habič *et al.*, 1989). These both have walls that are a combination of bare rock cliffs, undercut scars and very steep rocky slopes that now support a dense cover of trees (Figure 3.1.2). They both formed by collapse into complexes of large passages and chambers along the underground river course. High-level galleries were abandoned and undercut by newer river passages, and the Velika sinkhole is also at the junction of a major old branch passage towards the north. The dimensions of the sinkholes are the same order of magnitude as those of the largest chamber, 145 m long and 125 m wide, that survives intact downstream in the Skocjan cave (off the map, Figure 3.1.1) beneath a roof that is 100 m thick. It is not known how large were any individual chambers that suffered massive roof collapse, but the two sinkholes most probably developed by progressive roof and wall collapses into multiple passages. Lisicna is a third large

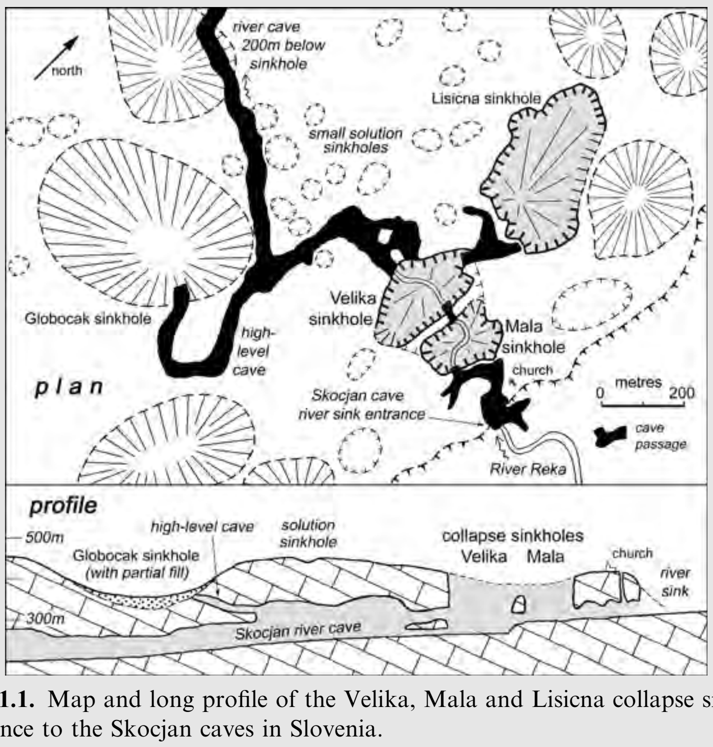

Figure 3.1.1. Map and long profile of the Velika, Mala and Lisicna collapse sinkholes at the entrance to the Skocjan caves in Slovenia.

collapse sinkhole, that appears to be failed into an old outlet cave passage whose northern continuation is hidden by collapse debris. Now lacking its cave river, it is a little more degraded, with a floor that is an inverted cone in a debris fill, but it still has cliffs around most of its perimeter.

The karst plateau above the Skocjan cave is pitted with numerous large and small solution sinkholes (Mihevc, 1998). One of the largest is the Globocak sinkhole, 500 m across and 90 m deep (Figure 3.1.1). A high-level cave passage in Skocjan is blocked by breakdown debris (but breached by a small mined tunnel) under its slopes, and appears to be an old trunk passage that once continued further west. The Globocak sinkhole is almost conical except for its aggraded debris floor, with no cliff margins that compare it to Velika and Mala. It appears to be a solution sinkhole, where collapse of the cave has made only a modest contribution to its development. The topography immediately around Velika and Mala suggests that a shallow solution sinkhole was developing above the caves, and its deepening contributed to the thinning of the limestone, and therefore to the ultimate collapses. The timing of this event, or of these events, remains unknown, but is not within the age of historical records.

Figure 3.1.2. The view eastwards across the Velika and Mala collapse sinkholes. The village houses and church stand above the cliff that drops into the Mala sinkhole breached by the upstream segment of the Skocjan cave, and the wooded ridge in the foreground separates the two sinkholes.
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Figure 3.10. A small collapsed cave in Penyghent Gill, in the English Pennines, where a roof about 1 m thick has collapsed along the length of a low cave that was 5–10 m wide.
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Total roof collapse over a length of shallow cave passage can create a very elongate collapse sinkhole lined by large slabs of undermined rock (Figure 3.10); failure of these wide caves in Penyghent Gill has created the largest area of undisturbed cave collapse in Britain (Waltham *et al.*, 1997). On a much larger scale this process can create a gorge or ravine (Figure 3.11); the Patale Chhango gorge in

Figure 3.11. The chaos of large fallen blocks that defines the collapsed cavern at the mouth of the Patale Chhango cave in lowland Nepal; the short gorge is 55 m deep, and most of the collapse blocks are of the conglomerate that roofs the visible cave openings.
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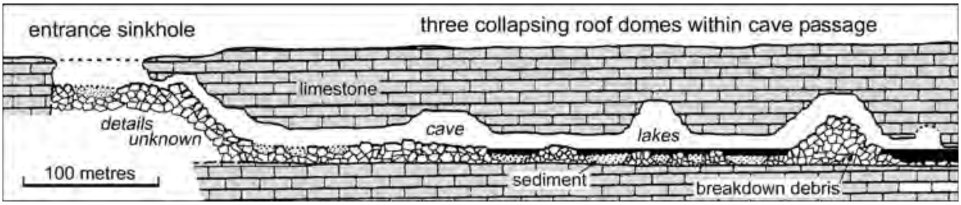

Figure 3.12. Long profile of Koonalda Cave, in Australia's Nullarbor Desert, with roof collapse forming the three high chambers and the entrance sinkhole; the rock floor of the main passage is unseen and its position is therefore conjectured, and the breakdown pile obscures any rock details below and left of the entrance sinkhole.
After survey by J. Hinwood *et al.*

Nepal is one of the very few in the world that have been created by massive cave collapse (Waltham, 1998). Nearly all karst gorges that are popularly described as "collapsed caverns" are actually subaerial fluvial features.

A collapsing chamber within a cave evolves into a collapse sinkhole when the original cave roof is thinned to the point of failure. This may develop entirely by roof stoping within the cave, or may be a feature of gross surface lowering. Both processes are natural and automatic in active karst terrains, and most collapse sinkholes owe their evolution to a combination of both processes. Roof stoping has largely or entirely formed the wide sinkholes into isolated large caves in the almost static surface environment of the Nullarbor Desert in Australia (Lowry and Jennings, 1974). Koonalda Cave has a number of breakdown domes along its large trunk passage, whose original dissolutional roof is about 60 m below ground (Figure 3.12). One dome is now within 15 m of the ground surface, and another has broken through to create the large collapse sinkhole at the entrance. In contrast, more than 20 large collapse sinkholes in the karst of Belize have been formed by surface denudation breaching an old high-level series of large cave passages, while large stable chambers still lie in the modern river caves at greater depths (Miller, 1987). It is also significant that the roof of a collapsing chamber may be thinned to failure by deepening of a solution sinkhole directly above it, guided by the same fracture zone. This is recognised as significant over Slovenia's Postojna Jama (Šebela, 1996), where deep solution sinkholes overly some of the cave's collapse chambers while large collapse sinkholes appear to overlie passage continuations now blocked by breakdown debris. The double process may also account for the formation of tiankengs (see Section 3.3.2).

The critical roof thickness for collapse was investigated by numerical modelling of the chamber that is 130 m wide in the cave of Brezno pri Medvedovi Konti in Slovenia (Kortnik, 2002). Local failures started to appear when the roof was thinned to about 20 m. The roof developed major failures when it was down to 6 m thick, and it had fully collapsed before the thickness reached 4 m. The collapse was modelled twice, with values of 1.85 and 4.0 MPa for the tensile strength of the limestone. Though pre-failure displacements were greater for the weaker material, the increase in minor failures and the ultimate collapse occurred at the same roof thickness for each material. The results broadly conform to field observations of cave chambers in the region, but are only regarded as preliminary until joint patterns are better incorporated in the numerical models. A programme of numerical analyses of flat-roofed caves under load indicated that caves about 50 m wide may collapse without any imposed load at depths of about 10 m in typical karst limestone, while caves 30 m wide collapse naturally when their roof is reduced to about 2.5 m thick (Section 7.2.2).

The Bahamian island of San Salvador has numerous small collapse sinkholes that are known as banana holes, after the crop that is grown so easily on the soils capping the collapsed rock debris on their floors. They are mostly <12 m across and < 4 m deep with vertical or overhanging walls, but lack accessible openings into continuing caves. The small chambers originally formed in the zone of preferential dissolution at the top of a freshwater lens graded to an old higher sea level (Wilson *et al.*, 1995). Where the roof rock was only about a metre thick it failed with the help of subaerial fissure development and dissolution from above, but it is thought that many other similar caves at slightly greater depth present a significant collapse hazard.

3.3.2 Tiankengs

The very largest collapse sinkholes are also known by the Chinese term, tiankengs (meaning *sky holes*). They are typically more than 250 m deep and wide, with vertical walls forming a large part of their perimeters, and the finest examples lie in the very mature karsts of China and New Britain. China's Xiaozhai Tiankeng is 660 m deep and 600 m across, with vertical walls above and below a sloping terrace at mid-depth (Figure 3.13); below the terrace a debris cone on one side of the hole descends to the river that crosses the floor of the tiankeng between cave passages only about 20 m wide (Senior, 1995; Zhu and Zhang, 1995; Zhu *et al.*, 2003).

The most likely origin of a tiankeng appears to be the deepening of a solution sinkhole directly above the stoping of a cave roof. Both features are likely to develop on a zone of locally increased rock fracturing. Deepening of the sinkhole concentrates infiltration, and thereby accelerates roof stoping below, where the cave river efficiently removes the breakdown debris. This model (Figure 3.14) was first based on the giant sinkholes of the Nakanai karst in New Britain (Maire, 1981), and may be applied on a smaller scale to many collapse sinkholes. Once its vertical walls are exposed, a tiankeng would evolve towards a rounded plan-form by face retreat due to weathering and small-scale breakdown. Distinct from this collapse origin, a second type of erosional tiankeng has been proposed (Zhu, 2001) where a large sinking river initiates the shaft development. However, waterfall retreat in a river sink tends to create an elongate slot that lacks the rounded form of the larger tiankengs, and waterfall erosion would appear to be only contributory to the collapse process. Tiankeng evolution is demonstrated by three sites within China's Xingwen karst (Waltham *et al.*, 1993). Xiaoyanwan is a massive collapse sinkhole ringed by vertical walls that truncate giant cave passages, while Dayanwan is an older degraded tiankeng, and the large chambers of Zhucaojing will eventually coalesce and collapse into a third tiankeng (Figure 3.15).

Figure 3.13. The giant collapse sinkhole of Xiaozhai Tiankeng, near Chongqing, China; the footpath winds across the wooded ledge before descending 300 m down a talus slope in the inner shaft.
Photo: Zhu Xuewen.

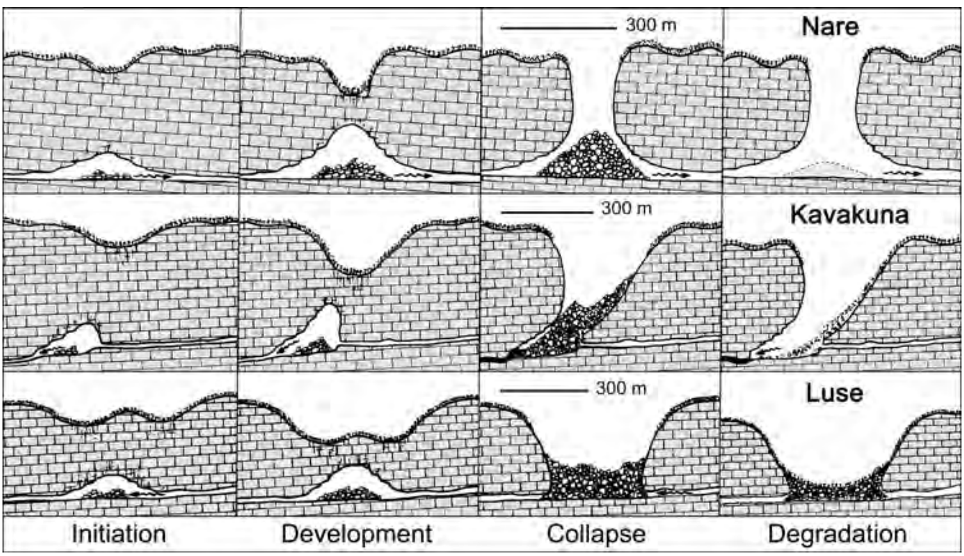

Figure 3.14. Four stages in the conceptual evolution of three of the tiankengs in the Nakanai karst of New Britain.
After Maire (1981).

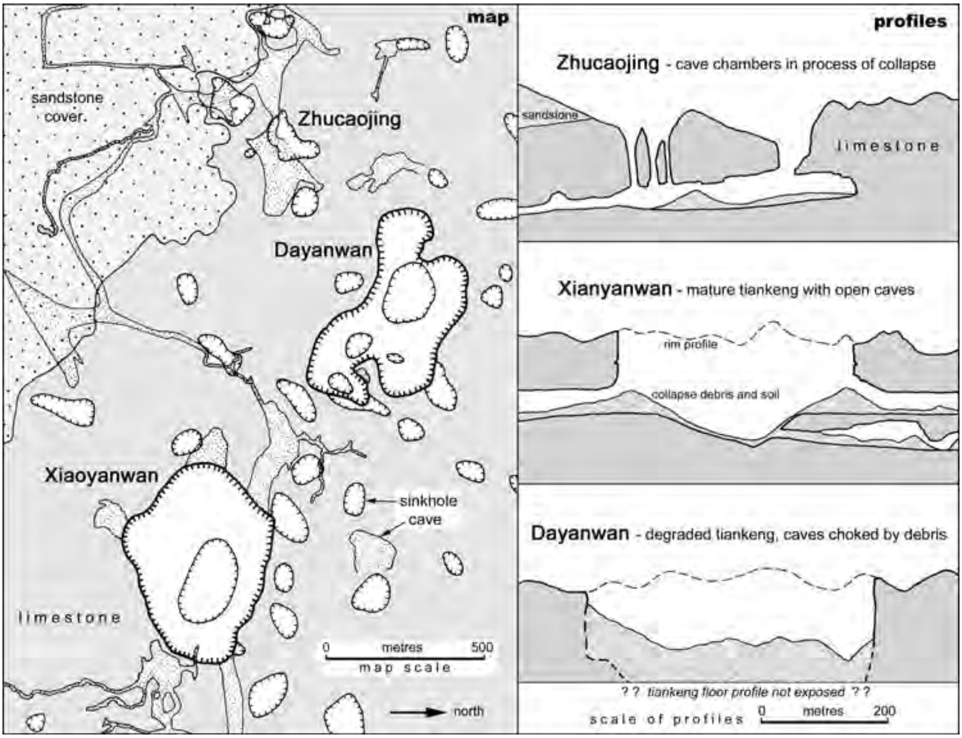

Figure 3.15. Map and profiles of the two tiankengs and the Zhucaojing cave that will eventually collapse into a third tiankeng in the Xingwen karst of China.
After surveys by the China Caves Project.

Tiankengs are essentially limited to the tropical terrains, but the mature karst of Croatia does contain some very large collapse sinkholes. In the hillside above the Imotski polje, the adjacent sinkholes of Crveno Jezero, 518 m deep, and Modro Jezero, 288 m deep, are each about 300 m across and half full of water; both these and the Skocjan sinkholes in Slovenia (Box 3.1) could be described as tiankengs. In Mexico's Sierra Madre Oriental, the Sotano de las Golondrinas is a huge bell-shaped shaft, over 500 m deep with breakdown debris 100 m deep across its floor that is over 200 m in diameter. Both Golondrinas and a number of other giant sinkholes in the same karst are exceptionally large isolated voids that lack associated river caves, and they may owe their initiation to hydrothermal sulphuric acid corrosion. Subsequently, they have been modified by roof stoping so that they now appear very similar to the classical tiankengs in China.

3.3.3 Collapse sinkholes in chalk

Collapse sinkholes are rare in chalk, as the weak rock contains few large cave passages. Though small-scale collapse does occur, it is generally subordinate to dissolution during landform evolution in chalk karst. An even more rarely observed collapse event created the Bîme des Enfants in France's Aube department in January 1948. It left a sinkhole 16 m deep and 10 m across, with walls in typically well-fractured chalk, in a gently sloping hillside (Figure 3.16). There was no cave accessible around the edge of the mound of fallen debris, but the immediate area is distinguished by having some long stream caves with passages generally 2 m high and 1 m wide, which are large by the standards of the European chalk. The limestones of Australia's Nullarbor karst are described as chalky, and have porosities normally >25%, but they do have large cave passages slowly developed in the desert climate, and consequently have large collapse sinkholes totally unlike anything in the European chalk.

A special feature of chalk is its susceptibility to liquefaction when it is both disturbed and saturated (Burland *et al.*, 1983). Disturbance may be due to engineering handling, when wet chalk is well known to liquefy into a slurry, but may also be due to stress within a cavity roof, or due to frost shattering. The latter is ubiquitous across the outcrops of southern England and reached depths of up to 10 m during the Pleistocene cold stages. Saturation is easily achieved at point inputs of drainage off a soil cover or by concentrated engineered drainage, and increased dissolution is then an inevitable side effect. A site with drainage from soakaway drains directed into chalk that has inherited frost shattering and stands over ground cavities is optimal for liquefaction and consequent ground failure. The well-known collapses (Figure 3.17) in a new housing estate at Bury St. Edmunds, U.K., remain one of the most spectacular consequences of chalk liquefaction (Waltham, 1989; Bell *et al.*, 1992). However, these developed around drains over old flint mines close to the depth limit of Pleistocene frost action, and strictly they are therefore crown holes. Comparable liquefaction over natural cavities is limited by the scarcity of adequately large caves, though sinkholes of this type have been known to develop over gull cavities along cambered escarpments. It is debatable as to how much liquefaction contributed to the roof stoping that formed the Bîme des Enfants (Figure 3.16). There were no indications of point drainage input that could have locally saturated the chalk, but the failure was during a cold and wet winter, and its exposed walls are in putty chalk and rubble chalk very similar to that in the collapses at Bury St. Edmunds.

3.3.4 Collapse sinkholes in gypsum

Due to the relative lack of gypsum at outcrop, collapse sinkholes in gypsum are not very widespread, as opposed to the abundance of caprock sinkholes over interstratal gypsum karst (Section 3.4.2). However their role as a geohazard is increased by their potentially rapid development under gypsum's high rate of dissolution. New collapse sinkholes occur almost annually in the forested gypsum karst at Pinega in northern Russia (Waltham and Cooper, 1998). Houses have been damaged in Rapid City, South Dakota, by the sudden development of collapse sinkholes up to 10 m across that have formed in the brecciated outcrop zones of gypsum beds up to 9 m thick (Rahn and Davis, 1996). Just over the state line into Wyoming, the Vore Buffalo Jump is a much larger collapse sinkhole in the same gypsum. Nearly 60 m across and

Figure 3.16. The collapse sinkhole of the Bîme des Enfants in the chalk karst of northern France, 26 years after its sudden appearance.
TW.

Figure 3.17. One of the collapse sinkholes formed by liquefaction failure of the chalk over old mines in Bury St. Edmunds, U.K., seven years after their failure destroyed the road.
TW.

20 m deep, it was used by native Americans around 400 years ago to stampede buffaloes to their deaths over its precipitous walls, but it is now degraded to a rocky bowl (and partly filled by thousands of buffalo bones) adjacent to the interstate highway.

Small collapse sinkholes are lost within the rapid denudation of gypsum outcrops, but larger features survive, especially in dry climates where dissolution is reduced. The nine "Bottomless Lakes" at Roswell, New Mexico (Martinez *et al.*, 1998), lie in sinkholes 50–100 m wide ringed by walls of broken gypsum up to 40 m high above water that is actually only 5–25 m deep (Figure 1.11). The sinkholes are cut into a dissolutionally cambered escarpment, and were largely formed by undermining and collapse around sites of rising artesian water (a process that is limited in limestone karst by the need for meteoric carbon dioxide to provide dissolutional aggressivity).

Isolated collapse sinkholes are scattered across the exposed gypsum karst near Sivas in eastern Turkey (Waltham, 2002a). Some of these are up to 400 m across and 50 m deep; they are orders of magnitude larger than gypsum caves observed locally or anywhere else. One sinkhole is still very active, as it is partially flooded with water

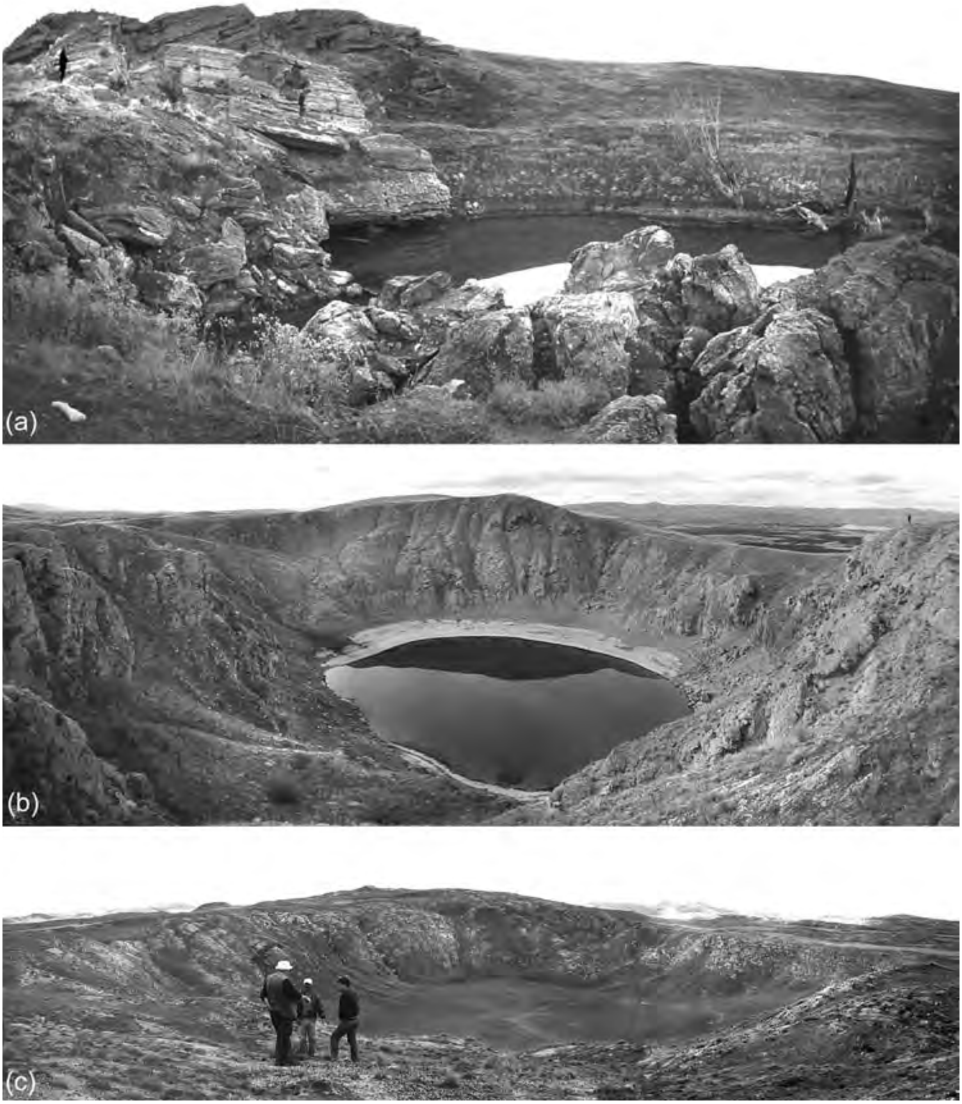

Figure 3.18. An evolutionary sequence of collapse sinkholes in the gypsum karst near Sivas, Turkey. (a) Active dissolutional undermining and rock collapse in a small corner of the riverside Bielekbasi sinkhole. (b) An almost static lake in the mature Kizilcam sinkhole. (c) A large degraded sinkhole on the hills above Mahmutaga.
TW.

that circulates rapidly to and from the adjacent Kizilirmak River. Its margin is an almost complete line of dissolutional undermining with small collapses producing talus slopes of gypsum blocks into the lake on its floor. In one corner, the collapse is into larger blocks (Figure 3.18(a)), and indicates a maximum failed span that could have reached about 25 m – compatible with the maximum sizes of gypsum caves recorded in many karsts. It appears that numerous small collapses accumulated to form the large Sivas sinkholes. An evolutionary sequence can be recognised (Figure 3.18) from the active riverside collapses, to the mature collapse sinkhole of Kizilcam far from the river's circulation and therefore less active, to the unnamed degraded sinkholes high on the plateau and fossilised by fluvial rejuvenation and water table decline. The same sequence may also apply to collapse sinkholes in limestone, but over much longer timescales.

3.4 CAPROCK SINKHOLES

Where roof stoping and cavern collapse migrate up through overlying non-karstic rocks, any ultimate failure of the surface creates a caprock sinkhole. These sinkholes are totally dependent on the initial formation of significant voids within the interstratal karst that is developed in the underlying soluble rock. Like collapse sinkholes, many caprock sinkholes have steep rocky walls as their surface features typically develop by rapid or instantaneous failure events. They may be only a few metres across, as is the example at Dankivsky (Box 3.2). However, they can be much larger, and the two examples of Dankivsky and Skocjan (Box 3.1) lie at opposite ends of the spectrum of sizes that can be found in both collapse and caprock sinkholes. The steep walls of fresh caprock sinkholes degrade to lower profiles over time, especially in the weaker caprocks, and the morphology of caprock sinkholes in poorly consolidated clay rocks is very similar to that of dropout sinkholes formed in weak but cohesive clay soils (Chapter 4). In plan shape, most caprock sinkholes tend to be more circular than most collapse sinkholes in exposed karst. This is because the stress-controlled stoping processes evolve as the void migrates up through the caprock, and smooth out any fracture pattern irregularities that guided dissolution in the original cave beneath.

Though caprock sinkholes may have diameters up to many hundreds of metres, their depths are generally limited by the lack of dissolutional removal of the breakdown debris. Their debris piles may reach considerable depths, and their morphology grades into that of deep-seated breccia pipes, but few caprock sinkholes are open to more than 20 m deep. Collapse features of larger areal extent in caprock are recognised as grabens and areas of foundered strata where the interstratal karst beds have been totally removed by dissolution, but these cannot be described as sinkholes.

3.4.1 Caprock sinkholes over limestone

The dependence of efficient carbonate dissolution on soil-derived carbon dioxide restricts extensive development of interstratal karst in limestones beneath cover rocks of low permeability. The optimum situation is a gently dipping limestone bed with drainage from an outcrop supplied by allogenic streams that pass beneath a tilted plateau of cover rock and out to a deep valley. This creates potentially large caves where collapse can be initiated. If the caprock is a permeable

Figure 3.19. A large caprock sinkhole over the interstratal karst of Mynydd Llangattwg in Wales, U.K.; all the exposed and collapsed rock is strong gritstone, and the top of the limestone is about 10 m below the floor of the sinkhole.
TW.

sandstone, any localised concentrations of downward infiltration to the buried limestone provide sites for collapse initiation, that ultimately stope upwards to create the caprock sinkholes.

Dip slopes of gritstone in southern Wales provide Britain's finest interstratal karst, expressed by hundreds of caprock sinkholes (Thomas, 1963, 1974). Mapping of the grit outcrops revealed 437 sinkholes, with average dimensions of 29 m across and 7.5 m deep (Figure 3.19). Small cavities are bridged by the strong grit, which therefore lacks the profusion of smaller sinkholes that characterise limestone outcrops. In contrast, there are numerous subsidence depressions and outcrops of foundered strata, each 100–300 m across, caused by dissolutional removal of the buried limestone in areas much larger than those causing the sinkholes. The maturity of the caprock sinkholes is reflected in more than a quarter of them having almost perfectly circular outlines, and also by aprons of talus and soil that drape most of the gritstone walls forming their perimeters.

Only in some of the Welsh interstratal karst are there known cave systems that can be correlated with the caprock sinkholes. Most of the sinkholes appear to have been caused by dissolution of the limestone from just beneath the permeable but insoluble grit cap, while some appear to relate to collapse zones and boulder chokes in large old caves far below. Sinkholes on the Llangattwg moorland appear to form an evolutionary sequence, and clastic cave sediments derived from the caprock grit are evidence of connections between the sinkholes and the caves (Bull, 1980). A caprock sinkhole, first developed by dissolution at the top of the limestone, may direct drainage towards a deeper cave, and thereby promote roof collapse that may in turn stope upwards to deepen the sinkhole by deep-seated undermining. This

BOX 3.2. CAPROCK SINKHOLE

An example – Dankivsky, Ukraine

The world's most cavernous interstratal gypsum karst underlies the Dniester Valley of the western Ukraine and northern Moldova. On 11 January, 1998, a caprock sinkhole developed in a clay hillside at Dankivsky when a small instantaneous surface collapse broke through from an underlying gypsum cave. The new sinkhole was 5 m across, with vertical sides that descended 22 m to a sloping floor of fallen debris. It was in the previously unbroken grass cover of a field (Figure 3.2.1), and the collapse was heard at a farm one kilometre away. At the foot of the newly opened shaft (Figure 3.2.2), there was an opening over the debris cone and into the domed roof of a cave 9 m wide (Fig. 3.2.3). This dome had formed by wider collapse of the gypsum roof beds of a cave passage that continued about 5 m high but totally underwater where its roof was intact. The cave passage is very similar to those, just 12 km to the south, that extend for 92 km in the maze cave of Zolushka, which has been drained by pumping activity in an adjacent quarry (Klimchouk and Andrejchuk, 2003). Both the gypsum and the caprock clays are of Miocene age.

Figure 3.2.1. The freshly collapsed clay walls of the Dankivsky caprock sinkhole, Ukraine, with the way into the underlying cave lost in the darkness.
Photo: Alexander Klimchouk.

Figure 3.2.2. The grass-covered clay slope at Dankivsky broken by the new caprock sinkhole.
Photo: Alexander Klimchouk.

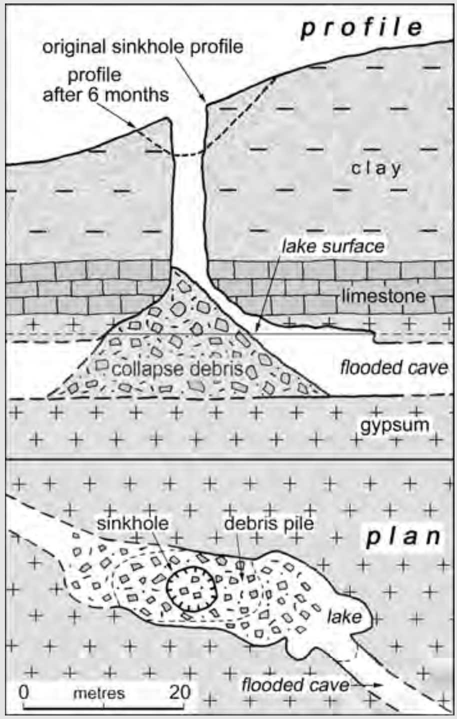

Figure 3.2.3. Plan and profile of the Dankivsky caprock sinkhole; the extensions of the cave underwater and behind the debris cone are unknown.
After Klimchouk and Andrejchuk (2003).

The roof dome at Dankivsky had developed on a slickensided fault zone that had provided a focus for upward dissolution of the gypsum, and probably also a site of downward percolation of water through the cover rocks. A nearby surface stream lies 19 m above the standing water level in the cave, indicating the presence of groundwater perched above the gypsum. By a combination of dissolution and collapse, the cave roof dome enlarged upwards through 3 m of gypsum. It then stoped upwards through 6 m of limestone and 17 m of clay before breaching the surface. Fallen debris from the shaft walls blocked the way into the cave four months after the initial collapse, and the shaft had degraded into a bowl-shaped sinkhole just 4 m deep within nine months of the event. Though the Dankivsky sinkhole is quite small, it demonstrated the morphology of both a fresh caprock sinkhole with vertical sides, and also a degraded caprock sinkhole that is barely distinguishable from other types except for its occurrence in the outcrop of an insoluble rock. Because the upper caprock is a clay, it also resembles a dropout type of subsidence sinkhole.

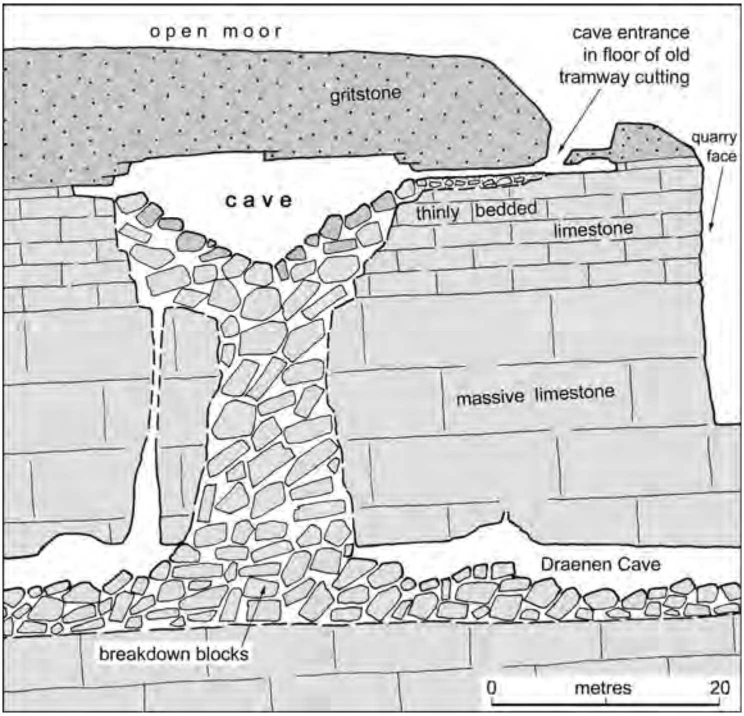

Figure 3.20. Profile through Siambre Ddu, a cave that has already migrated from the limestone up into the overlying gritstone and will ultimately form a new caprock sinkhole in the interstratal karst of South Wales, U.K.
Draenen passage after survey by J. Stevens *et al.*

process accounts for the Waen Rûdd sinkhole, 100 m above a collapse in the cave passage, but caprock sinkholes are unrelated to collapse zones in the caves at greater depths.

At the eastern end of Wales' interstratal karst, the cave of Siambre Ddu appears to offer a model for formation of the caprock sinkholes. It has a single rounded chamber, 25 m across and 9 m high, with walls and roof entirely in the gritstone (Figure 3.20), entered through a low collapse-modified passage that formed in the top bed of the limestone. The roof is about 6 m thick, and there is as yet no surface sinkhole. Breakdown blocks of grit line a subsided cone in the floor of the chamber, which descends well below the original limestone/gritstone contact. About 20 m below, a passage 10 m wide in the cave of Ogof Draenen is nearly blocked by a cone of gritstone blocks that descend from totally choked shafts (Figure 3.20); an adjacent clean shaft carries drip-water from Siambre Ddu. Any collapse zone that may have stoped upwards from the lower cave is obscured by debris, but the massive lower limestone is more likely to contain just the dissolution shafts that are now tapping the pile of gritstone breakdown blocks from below. It appears that Siambre

Ddu originated by dissolution forming a wide cavern near the top of the thinly bedded upper limestone, and it will ultimately develop into a caprock sinkhole.

Britain also has numerous sinkholes in the weak sedimentary rocks that provide a caprock on parts of the chalk dip slopes (Sperling *et al.*, 1977). These are better described as subsidence sinkholes (Chapter 4), as there is no evidence of collapse into large cave chambers in the chalk, and the poorly consolidated sands and clays of the cover appear to have behaved as soils that have been washed down into networks of fissures within the bedrock.

3.4.2 Caprock sinkholes over gypsum

Interstratal karst is extensive in gypsum that is highly soluble in groundwater not dependent on subaerial processes for its aggressivity. Dissolution of gypsum buried at shallow depths ultimately causes vast numbers of caprock sinkholes, and dissolution at greater depths creates many deep-seated breccia pipes (see Section 3.5).

Sinkhole densities on interstratal gypsum karst can rise to 200 per km^2, with new sinkholes appearing at rates of 0.01 to 3.0 per km^2 per year. These are mainly caprock sinkholes but include significant proportions of subsidence sinkholes where the gypsum is capped by thin and poorly consolidated sedimentary materials that behave as soils. The best-documented cases are in the Ukrainian karst, where the sinkholes can be correlated with breakdown features in extensive networks of accessible caves (Klimchouk and Andrejchuk, 1996, 2003). Roof failures in the caves develop mainly at drainage points (mostly inlets, but also where outlets existed before the cave was drained by local rejuvenation), which are mainly on joints or faults. Most passages are about 3 m wide, and wider chambers are not major collapse zones as they are mostly formed in stable zones of less fractured rock. Roof stoping then proceeds through the upper gypsum and a thin limestone, and then through the capping clays and sands until a collapse sinkhole develops (Figure 3.21). The sinkhole may initially be steep-sided (Box 3.2), and later degrades to a gentler bowl, or it may start with a lower profile where liquefiable sands in the cover sequence fail by suffosion instead of collapse (Chapter 4).

Caprock sinkholes decrease in numbers where the caprocks are thicker. The critical cover thickness relates to the material properties of the cover and the sizes of the caves, and is specific for each region. In Ukraine, critical thicknesses beyond which sinkhole densities decrease noticeably range from 20 to 40 m. The passages of Mlynki cave contain 144 roof breakdown features, but there are only two sinkholes above, as the cover is a clay 25–30 m thick. There is a close correlation between sinkhole distribution and the underlying Kungur Caves, in the Russian Urals (Figure 3.22), where the caprock is down to about 25 m thick, but sinkholes also exist in smaller numbers where the caprock is over 60 m (Klimchouk and Andrejchuk, 1996).

Roof collapse in interstratal gypsum caves creates columns of largely insoluble breakdown debris (Figure 3.21) that are small versions of the breccia pipes formed over sites of deep-seated dissolution of either gypsum or salt (Section 3.5). These features constitute a significant geohazard due to their potential failure at the surface

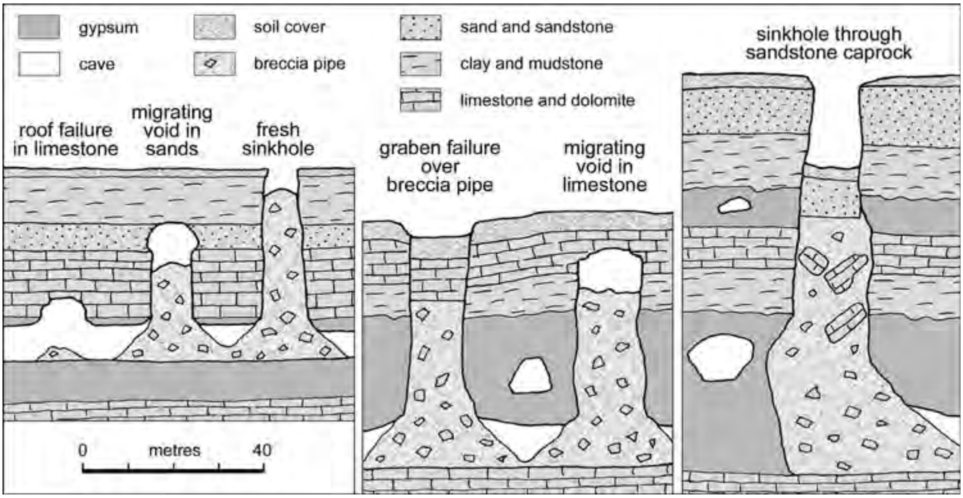

Figure 3.21. Evolving and maturing caprock sinkholes over the gypsum karst of Ukraine and England.
After Klimchouk and Andrejchuk (1996) and Cooper (1998).

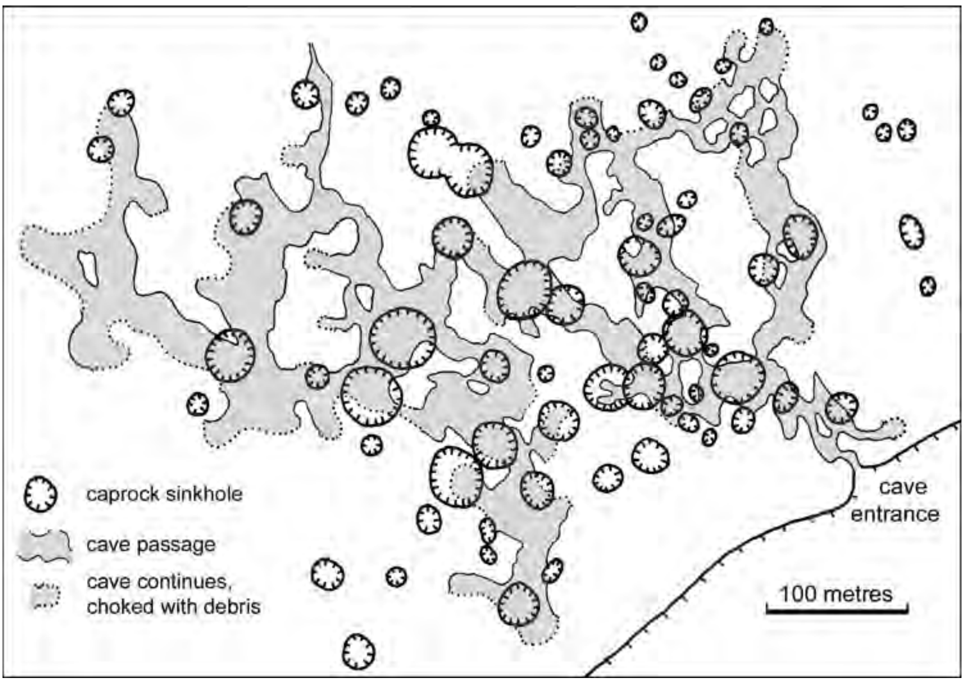

Figure 3.22. Correlation between caprock sinkholes and cave passages in the buried gypsum at the Kungur Caves in Russia.
After survey by K. Gorbunova *et al.*

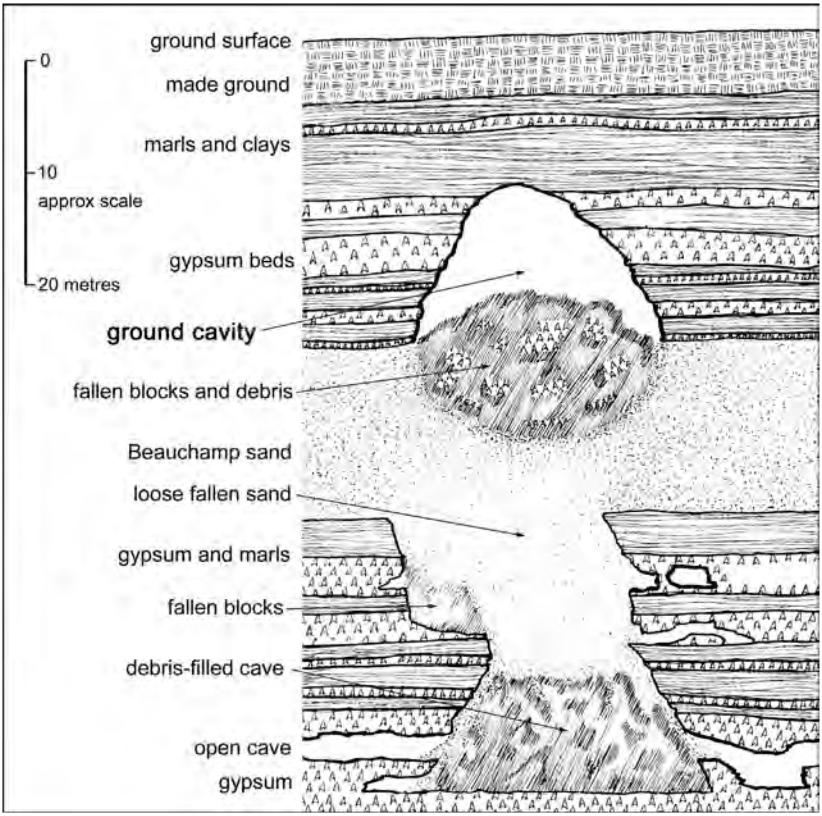

Figure 3.23. Schematic section through a concealed void of 2,500 m^3 found entirely within cover rocks over gypsum and beneath a Paris railway station.
After Toulemont (1987).

as caprock sinkholes, unless the bulking of the breakdown material fills the void and provides roof support. Many surface failures occur over the gypsum beneath Paris, France (Toulemont, 1987), and investigation of a cavity found in 1975 beneath railway engineering works revealed a failure migrating up through the cover rocks from dissolution cavities in a number of gypsum horizons (Figure 3.23). Numerous caprock sinkholes at Ripon, U.K., lie on top of breccia pipes that are reactivated by ongoing dissolution of the gypsum lying beneath 40–60 m of mudstone, limestone and sandstone cover (Cooper, 1998; Cooper and Waltham, 1999). Surface collapses are generally 10–30 m across, and cause extensive and repeated damage to the town's roads and buildings (Case study #1). Over time, most caprock sinkholes in weak cover rocks degrade to gentle depressions that may be confused with solution sinkholes unless the subsurface structure is investigated or exposed. Those in stronger rocks survive as deep circular pits, as seen at a number of sites in Canada where dolomite overlies gypsum within the Prairie Evaporite Formation under northern Alberta (Figure 3.24).

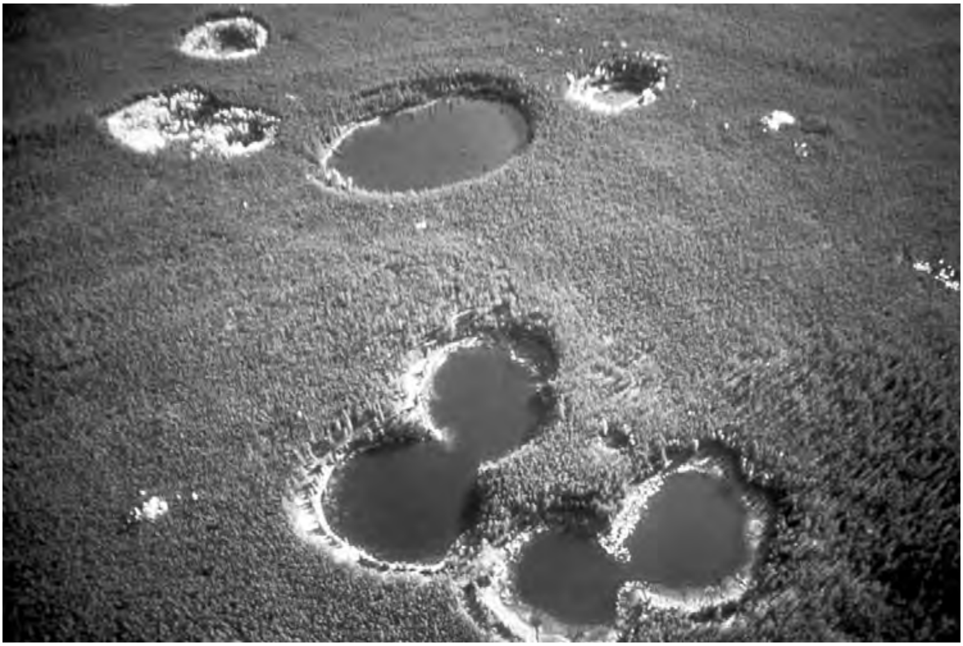

Figure 3.24. Caprock sinkholes rimmed by dolomite over the Wood Buffalo interstratal gypsum karst of northern Alberta, Canada.
Photo: Derek Ford.

3.4.3 Caprock sinkholes over salt

Once salt is reached by circulating groundwater, its dissolution can be both rapid and extensive. In some situations, caprock sinkholes can be formed, but total removal of the salt and consequent regional subsidence are more common. The McCauley Sinks, in eastern Arizona, include twenty caprock sinkholes that are each more than 100 m across and 30 m deep (Neal, 1995). They are formed in sandstone over thick salt beds at a depth of about 300 m, due to discrete cavity migration and breccia pipe development through a remarkable thickness of cover; there are no local indications of diapiric rises of soluble salt that could pre-date pipe formation. These sinkholes lie just ahead of a dissolution front that is migrating down dip and causing more widespread removal of the salt. Ultimately, the breccia pipes and their caprock sinkholes will be incorporated into the disturbed rock structure of a bowl of regional subsidence.

In March 1879, a sinkhole developed catastrophically near Meade, in western Kansas (Figure 3.25). It was 52 m across, and initially 27 m deep though filled with water to within 4 m of its rim. It overlies salt at a depth of about 150 m and is clearly a caprock sinkhole, though the surface feature may have been widened by slumping of the cover soils in the manner of a subsidence sinkhole. Significantly, the salt below

Figure 3.25. The caprock sinkhole that developed in 1879 over deeply buried salt at Meade, in Kansas.
Photo: US Geological Survey.

Meade is broken by faults that allow groundwater circulation to and from permeable sandstones and thereby encourage localised dissolution (Frye and Schoff, 1942). A century later, the Wink Sink formed in northern Texas (Case study #11). It was a caprock sinkhole 34 m deep and 110 m across, and the timing of its development is uniquely well known (Johnson, 1989b). The roof of a new cavern collapsed, and formed the sinkhole after void migration through 400 m of cover within less than 50 years – at a mean rate of at least one metre every six weeks.

Even more remarkable is the caprock sinkhole that developed in 1986 over a new breccia pipe above the Berezniki potash mine in the Russian Urals (Andrejchuk, 2002). Brine leakage into the mine, at a depth of 400 m, warned of massive dissolution of the 90 m of salt overlying the mined potash, though the extent of older cavities remains unknown. Only seven months later, cavity migration reached the surface through 300 m of limestones, mudstones and sandstones with the instantaneous appearance of a new caprock sinkhole 150 m deep and 40×80 m across at the top of a breccia pipe of the same plan dimensions. Roof stoping over the pipe took only about 12 days through the last 100 m of mudstone, but this very high rate was aided by structural weaknesses in a fracture zone along a local fold axis. The dissolution cavities in the salt were clearly very large, but the rate of cavity migration through the caprock has implications for ground stability in many terrains of interstratal karst.

While these caprock sinkholes have developed over breakdown sites that created breccia pipes, Crater Lake in south-east Sakatchewan, Canada, appears to have formed by block subsidence within ring faults (Christiansen, 1971). The sinkhole is 300 m across, largely filled by a lake with thick sediment above a rockhead that lies 30 m below the surrounding ground level. Salt in the Prairie Evaporite Formation lies at a depth of 900 m, and it is unknown to what extent a classic breccia pipe is developed at depth beneath the ring faults that breach the upper cover. Though the structure beneath Crater Lake may be more akin to that of a volcanic caldera, it is essentially a variety of caprock sinkhole, albeit an unusual variety.

3.5 BRECCIA PIPES

Collapse and stoping of the roof of a cavern created by dissolution can migrate upwards through considerable thicknesses of cover rock to create columns of fallen breakdown. These are generally known as breccia pipes. They may be only a few metres across and a few tens of metres deep, as are the debris piles that underlie all caprock sinkholes (and most collapse sinkholes); these features are effectively small-scale breccia pipes (Figure 3.21). But they may be much larger, extending up through many hundreds of metres of cover rocks, and proportionately many tens or hundreds of metres across. These deep-seated breccia pipes may or may not reach the surface. Where they do crop out, the initial collapse or caprock sinkhole may subsequently be degraded to a very modest surface depression. Alternatively, they may just appear as small circular breccia outcrops with no specific topographic expression. Settlement of the breccia within a pipe does constitute a potential subsidence hazard, but surface collapse events are extremely rare.

Breccia pipes are not a major feature of limestone karst. Strong limestone tends to develop stable arched roofs over caverns of the size normally created by cave river erosion. Typically, cavity migration is terminated with an arched roof over a modest pile of breakdown within a chamber only slightly modified by the collapse process. Exceptions do occur in thinly bedded or heavily fractured limestone. A breccia pipe within thinly bedded limestone, fortuitously exposed in an island cliff in Vietnam's Halong Bay, is at least 20 m high (its floor is below water level) and about 5 m across (Figure 3.26). It still has an open cavity on top of the column of limestone blocks, with about 5 m of overlying limestone ready to fail in order to create a collapse sinkhole. However the breccia pipe now appears to be inactive, with stalagmite growth above it, some cementation of the breccia and no signs of recent settlement within the pipe. Deep inside Switzerland's Hölloch cave, the Schwarzer Dom is the top, open part of a breakdown-filled shaft, 225 m high, with its roof still nearly 400 m below the ground surface (Figure 3.27). This style of rather short breccia pipe is only recognised in limestone caves where there happens to be access to both the top and foot of the debris pile, but it does demonstrate the process of progressive roof stoping in limestone.

Larger breccia pipes in limestone occur as features of paleokarst that have developed over very long timescales. These are generally associated with mineralisation, with the implication that dissolution by hydrothermal fluids may be essential to their development on this scale. A number of breccia pipes are exposed in the walls of the Grand Canyon, Arizona, with bases in dissolution zones in the Redwall Limestone, and rising through about 600 m of shales and sandstones, so that some reach the Kaibab Limestone at canyon rim level. Some younger caves have exposed the breccia pipes underground (Wenrich and Sutphin, 1994), and renewed settlement within a pipe 80 m in diameter has left an open cavern with walls of insoluble clastic

Figure 3.26. Limestone breccia pipe with an open cave at its top, exposed in the cliff of an island in Halong Bay, Vietnam.
TW.

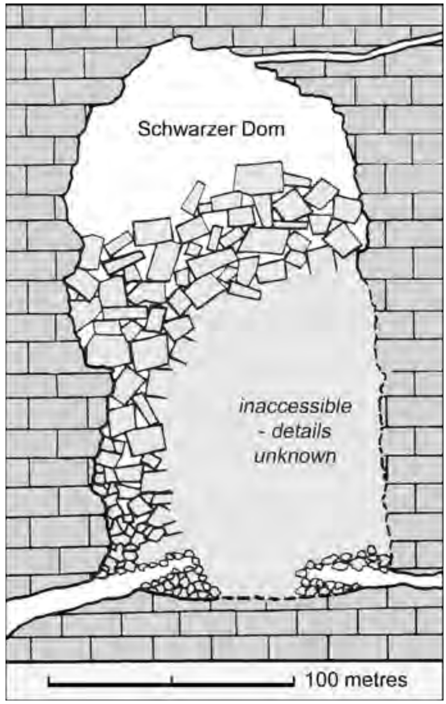

Figure 3.27. Cross section through the Schwarzer Dom in Hölloch, Switzerland, where progressive collapse of a cave roof has produced a short breccia pipe.
After Bögli (1980).

rocks in Paiute Cave (Hose and Strong, 1981). The roof of this has then stoped obliquely into the wallrock, to create an entrance through a collapse sinkhole in the Kaibab Limestone. The nearby Ah Hol Sah is a collapse sinkhole, 150 m across and 50 m deep, in the Kaibab Limestone outcrop. It appears to overlie another deep-seated pipe, but its breakdown has destroyed or obscured any exposures of breccia.

Nearly all very deep breccia pipes originate from dissolution of gypsum or salt. These materials can host more extensive cavities, that are formed more rapidly by deeply circulating groundwater at greater depths than comparable features in limestone, and they also lack limestone's strength to span the larger voids. It is estimated that there are more than 5,000 breccia pipes over gypsum and salt in North America (Quinlan *et al.*, 1986). Diameters range up to 1,000 m, and they propagate from depths as great as 1,200 m, but the total number includes features down to just 1 m in diameter that are more comparable with the cave roof collapses and caprock sinkholes described in Section 3.4.2.

Probably the finest exposures of deep-seated breccia pipes have been achieved in the coal mines and boreholes that intersect them in China. Some 2,875 breccia pipes

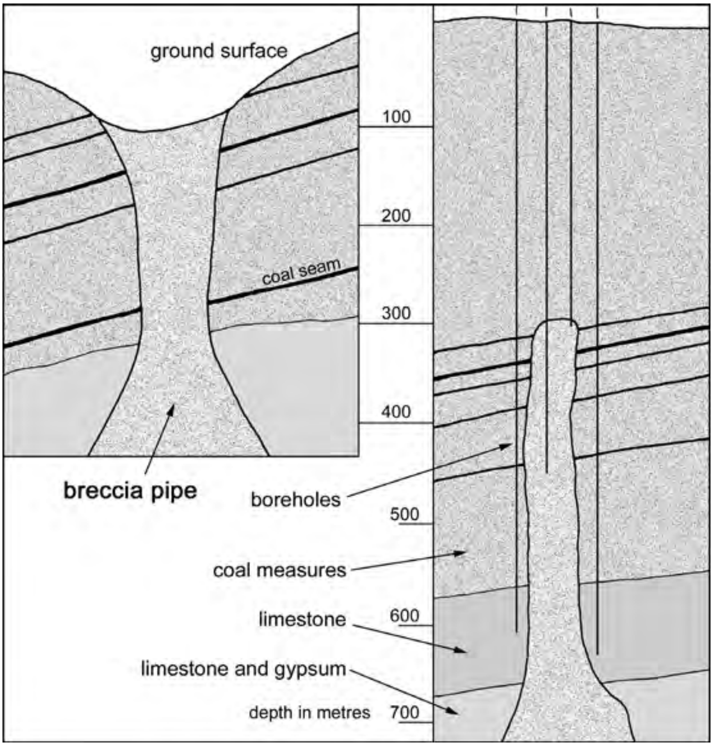

Figure 3.28. Two large breccia pipes exposed in coal mines over interstratal gypsum karst in northern China.
After Lu and Cooper (1997).

have been recorded, mainly in the mines of Shanxi and Hebei (Lu and Cooper, 1997). They originate on dissolution cavities within Ordovician gypsum that is interbedded with limestone, and they pass up through the Carbo-Permian coal measures (Figure 3.28). The vertical transmissivity of the pipes creates a major mining hazard from large flows of water that pass either up or down them. The pipes vary in diameter from tens of metres to a few hundred metres, and the larger ones pass up through 500 m of cover. Some reach the surface and contain a loose upper fill, but these are paleokarstic pipes that have been breached by surface lowering, and they never were active caprock sinkholes. Others are topped by stable bedrock arches. Near the northern margin of the Shanxi coal basin, 1,300 pipes are known in the Xishan mine, with a pipe density reaching 70 per km^2. It is significant that the breccia pipes are located mainly around the margins of the coalfield, and are not known in the deeper centre of the basin.

Breccia pipes comparable to those in China have been encountered in mines in Germany, Belgium and Canada, and have been mapped at outcrop in many other countries. Large breccia pipes over salt have also been induced artificially by uncontrolled brining operations that have caused ground failure in large sinkholes. Though

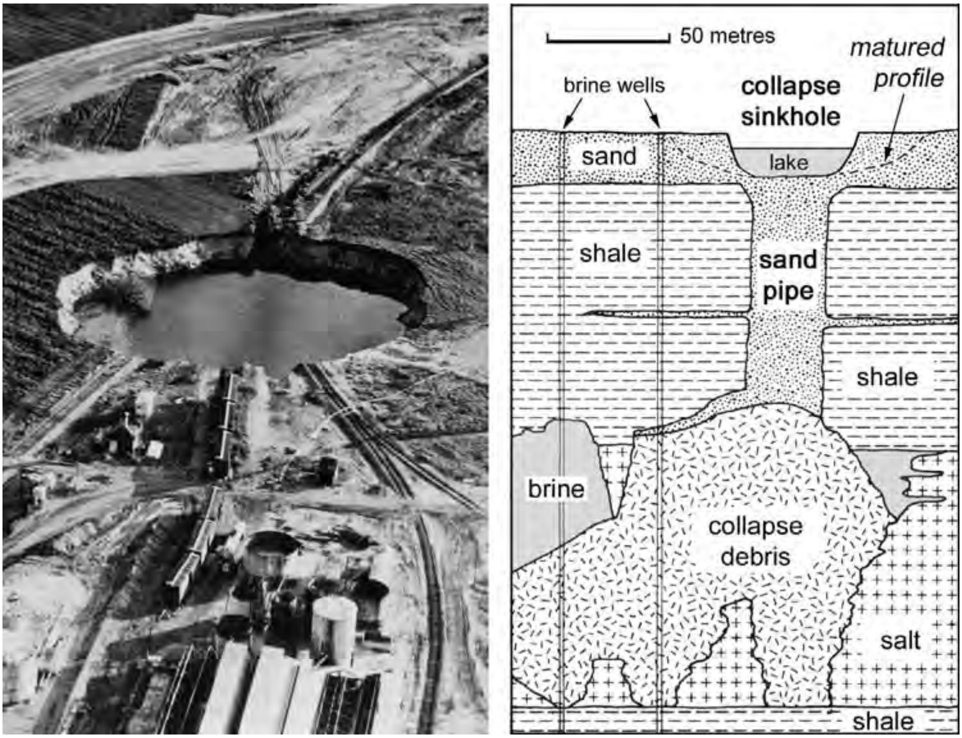

Figure 3.29. Oblique air photograph and conjectured profile of the caprock sinkhole that developed over a breccia pipe rooted in a large void created by brine pumping from a salt bed 130 m below ground level, near Hutchinson, Kansas.
Profile after Walters (1997). Photo: Hutchinson News.

these are effectively mine roof collapses, the ground structures revealed by post-failure investigations (Walters, 1977; Wassmann, 1979) provide useful models for the development of breccia pipes and caprock sinkholes (Figure 3.29). They are comparable with the Wink Sink in Texas (Case study #11), whose failure through 400 m of cover in about 50 years provided a unique indication of long-term stoping rates from depth. However this stoping was through very weak rocks, and stoping rates in cavernous limestones are orders of magnitude greater.

3.6 THE COLLAPSE GEOHAZARD

Though collapse landforms are a characteristic of most karst landscapes and collapse processes are integral to the evolution of most caves, natural events that involve surface collapse of bedrock are extremely rare. Collapse and caprock sinkholes rarely occur at densities higher than one per square kilometre in limestone landscapes that have matured through perhaps a million years. In such situations, the chance of a new sinkhole developing within a single construction site of one hectare during a lifetime of 100 years is therefore one in a million. Small caves and collapses can achieve similar densities in young coastal limestones within timescales that are an order of magnitude less, implying an equally increased probability of new sinkholes (Wilson *et al.*, 1995). Either of these figures presents a very small engineering risk, which may be further greatly reduced by prior ground investigations to locate any hazardous voids close to the surface by geophysics and/or shallow boreholes. In most karst terrains, collapse and caprock sinkholes are much more widely spaced, and the hazard is reduced. Higher densities of these types of sinkholes are largely restricted to some very mature karsts where landforms have evolved over much longer timescales, and the implied geohazard is therefore comparable to that in less mature karsts. In all cases, the risk of ground failure induced by imposed load due to engineering works is slightly greater (Chapter 7). The extremely small geohazard from rock collapse is totally independent of the much greater hazard from subsidence sinkholes developed in any soil cover (Chapter 4).

Collapse and caprock sinkholes constitute a far greater geohazard on gypsum karst due to its faster dissolution processes. In the English town of Ripon, 43 events of subsidence or collapse in the caprock over the gypsum have been recorded in the last 160 years, within an area of about 7 km^2 (Cooper, 1998). This gives a mean rate of one new sinkhole every 26 years in each square kilometre. The highest event rates are found in the thin and weak clay caprocks above the interstratal gypsum karst of the Ukraine (Klimchouk and Andrejchuk, 1996), where new sinkholes appear at rates of 0.01 to 3.0 per year per km^2. This implies a chance of greater than evens that any misguided one-hectare construction site in the worst areas will develop a new caprock sinkhole during its lifetime of 100 years. Many of these events are induced by engineering activity, so the natural event frequencies are lower, but these figures are pertinent to sites where development is taking place. However, the implication remains that appropriate engineering can reduce the hazard (Chapter 11). Both the soft ground and the high hazard level in the Ukrainian karst approach the conditions of subsidence sinkhole development in soils (Chapter 4).

Collapse sinkholes, caprock sinkholes and breccia pipes that reach outcrop all offer ground conditions that are essentially columns of broken rock. Many of these offer a subsidence potential by settling and compaction, either under load, or by down-washing and suffosion of their fines, and/or by continuing dissolution of their breakdown blocks. These hazardous sites are normally small and easily identified, and are best avoided by construction where they occupy only a small proportion of the karst area.

7

Rock failure under imposed load over caves

An ever-present geohazard in karst terrains is the collapse of bedrock into open caves when engineering works inadvertently impose new loadings on the unsupported spans over unknown caves. However, rock collapse is a rare event. A scatter of collapse and caprock sinkholes exist across most karst terrains (Chapter 3), but their small numbers have developed through geological time, albeit without imposed loading. Nearly all collapses induced by engineering activity in karst are subsidence sinkholes that develop entirely within the soil profile (Chapter 8). Induced rock collapse may be rare, but events can be catastrophic and should be avoided by appropriate engineering design.

A widely used guideline figure for safe (or allowable) bearing on limestone is 4 MPa (British Standards, 1986), but this assumes sound rock and takes no account of large, unseen voids. Column loads in large structures may be 5–10 MN, which therefore bear on pads 2–3 m across and generally reduce imposed stresses to no more than 2 MPa. Small pads over potentially large caves are effectively point loads on the cave roof. Raft and strip foundations impose less critical loads, especially where they are reinforced to span potential voids. Pile tips generally impose stresses no greater than about 1 MPa, especially where loads are partially carried by skin friction through the soil and rock cover, but these can impose high stress concentrations over small footprints that are also effectively point loads. Safe bearing pressures are lower on weak limestones, and are generally taken as around 750 kPa on sound chalk, but caves tend to be smaller in such rock types. Potential settlement may have to be considered where heavy loads are imposed on some of the chalks, weaker limestones and evaporite rocks, even where cave hazards have been accommodated. Highways impose much smaller loads. In British highway design, overall imposed stresses at ground level are assumed to be 17 kPa, with an additional 33 kPa along a metre-wide strip at the worst-case site. However such stresses are generally distributed by the underlying soil, and by the roadbed, before imposition on the bedrock.

Structural design in cavernous karst has to allow for loading imposed directly over the largest cave likely to exist in the area. Maximum safe loads are therefore a function of the width of the cave and the thickness of its intact rock roof (beside the rock mass strength, which normally dictates the safe bearing pressures, cited above, for sound rock). This may be expressed as thickness/width, either as a percentage or as a cover ratio. A broad rule-of-thumb has been that integrity is ensured where roof thickness exceeds cave width. Though this may be applicable in weak limestones and chalk, it appears to be excessively conservative for typical cavernous karst in strong limestones – where safe roof thicknesses that are 50% or 70% of cave width appear to be more appropriate. There is then the problem of assessing the maximum likely size of a cave beneath any construction site – and this can be based only on local knowledge, or broadly by reference to the engineering classification of karst (Waltham and Fookes, 2003; Waltham, 2002b).

Any justification or refinement of the thickness/width cover ratios for construction over caves is limited by the difficulties of defining rock mass strength, especially within a thin cave roof that will be subjected to point loading and consequent flexural distortion. This is especially problematic where dissolutionally opened fissures have to be characterised within karst ground. The available literature, both in textbooks and academic papers, is minimalist on this subject and refers to "calculations by established principles of rock mechanics", while studiously avoiding any presentation of useful numbers. Even where ground conditions are well explored and tightly defined, data is scarce, but the question of "how thick is a stable rock roof?" requires attention.

7.1 RECORDED COLLAPSE SINKHOLES INDUCED BY LOADING

Though true rock collapses over caves are rare, some have occurred during construction works. At Dhahran, Saudi Arabia, a cave roof 2.5 m thick collapsed under the 50 tonnes load of a Caterpillar D9 bulldozer (Grosch *et al.*, 1987). The cave was then filled with 1,200 m^3 of concrete, but its width was not stated, so the cover ratio, in the weak limestone of the region, is unknown. Twice in 1995, heavy bulldozers dropped through cave roofs in basalt during construction projects on Hawaii. On these young volcanic islands, lava tubes about 2–10 m wide lie beneath rock roofs 1–8 m thick, so many are inherently unsafe, but no great damage has been done, and collapse details have not been recorded.

Four spans of a concrete bridge near Tarpon Springs, Florida, failed in 1969 when three supports dropped out of sight – in a rare case of infrastructure loss into a new, self-induced, collapse sinkhole. The piers were H-piles driven into the porous bedrock limestone, and their simultaneous failure suggest that this was due to the total collapse of a zone of bedrock as it fell into an underlying, unknown, flooded, karstic cavern (Sowers, 1975). Corrosion could not have accounted for such a total failure, but may have contributed by reducing skin friction on the piles and thereby increasing their end loads.

Highway loading is normally so low that roads are unlikely to induce failure in

Figure 7.1. Fractured rock around the margin of the new collapse sinkhole that destroyed Dishman Lane in Bowling Green, Kentucky, in 2002.
Photo: Hilary Lambert, KEEP.

bedrock, but an induced collapse sinkhole did drop one Kentucky road into an underlying cave chamber (Case study #2). The cave ceiling lay about 7 m below the road, but 2 m of this was soil cover, and the upper half of the rock was heavily fissured and pinnacled. The effective rock slab was therefore only 2 or 3 m thick, and it spanned a cave chamber over 25 m across. This represented a cover ratio of about 10% – at which roof failure was almost inevitable (Figure 7.1). It appears that the road itself did not provide the critical loading (and there was no heavy vehicle there at the time), but failure of the rock slab probably occurred under the impact loading of the road as it dropped when voids collapsed within the intervening soil profile.

There are numerous reports from around the world of "limestone collapses" into sinkholes, but most of these appear to be "ground collapses in limestone regions". Reports and accounts emanate from construction sites, from highways and from just a few completed structures, but when the sites are examined or reported in detail, they are nearly all found to be failures of soil over fissures and cavities within stable bedrock. Induced collapses of limestone, gypsum or basalt that span natural cavities may be rare, but they do offer a significant geohazard to the unwary engineer who might impose inappropriate loads on ground regarded as solid and safe.

7.2 BEARING CAPACITY OF CAVE ROOFS

Observation of cave ceilings reveals that very few roofs are formed by single unbroken slabs or beds of intact rock. They must therefore be regarded as fractured rock masses. However, rock mass strength is notoriously difficult to assess; it may be safely estimated where it can be seen in the roof of an accessible cave, but it is very difficult to quantify for an unseen rock mass within the ground that may be straddling an unseen cave. In typical cavernous karst, the limestone is strong (intact unconfined compressive strength (UCS) of 50–100 MPa), is massively bedded (with beds 0.5–5.0 m thick) and has a mean fracture spacing of 1 m or more, typically across three or more fracture sets. Many of the fractures are open and locally enlarged by dissolution, tectonic folding may mean the dominant bedding plane fractures are steeply dipping, and lateral confining stresses are normally low at shallow depths. These parameters dictate that typical karst limestone can generally be regarded as a rock mass at the weaker end of Class III, with Q of around 4 on the classification scheme of Barton *et al.* (1974), and RMR of about 40 on the rock mass rating scheme of Bieniawski (1973). Strong cavernous limestones in Tennessee have been assigned RMR of 50 to 65 (Siegel and McCracken, 2001). Of the other rocks that may contain caves, gypsum, chalk and the weaker limestones generally constitute rock masses of Class IV or V, while many basalts are of Class II (caution should be exercised when these classifications of rock mass, though very useful, are used for purposes other than as originally intended with respect to tunnel excavation and support). All these estimates of rock mass strength and class are open to adjustment, up or down, on inspection of the rock conditions at any specific site.

Structural analysis of a cave roof can treat the rock mass as either a beam under flexural stress or an arch in compression. Both cases must be examined with or without imposed loads that are either spread or applied over small areas. Strengths of rock masses are negligible in tension and flexure, but remain high in compression. Where a cave roof profile is analysed, calculated bearing capacities for rock beams are smaller than those calculated for compression arches within the same profile. This is at least partly due to the rounded profiles of most caves that leave marginal buttresses to support a beam that only thins over the crown of the cave. Some beam analyses indicate factors of safety of less than unity for caves that are still standing, and are therefore clearly wrong, probably due to inherent difficulties in estimating rock mass properties. Loaded cave roofs appear to derive their integrity from arch development within their profiles, and beam analysis appears to be unhelpful in their assessment. There are no known beam failures under imposed loading in the Miami Limestone of southern Florida. Even though this rock is typically weak and thinly bedded, and is structurally loaded *in situ* over loose, deformable sands (and not normally over open caves), full-scale foundation load tests have not achieved beam failures of the rock (Kaderabek and Reynolds, 1981).

7.2.1 Integrity of loaded rock arches

A rock mass is stable over a void where an arch, capable of carrying its own load and any imposed load, can develop within its profile (Figure 7.2). This structure may be known as a voussoir arch (derived from the French for a stone arch) where the entire load is carried in compression. A bridge of this form is built of an arch of stone blocks that carry all loads normal to their interfaces, and therefore requires no cementing material; the analogy to a fractured rock mass depends on the existing fracture pattern. Structural analysis indicates that an arch within rock normally fails in compression (Sofianos, 1996); shear failure at the buttress occurs only in very short spans, and buckling failure is limited to arches in very thinly bedded rock. Punching failure is not a function of arch structure, and is a hazard only in thin rock slabs (Section 7.2.3).

The bearing capacity of a voussoir arch increases with its thickness, as it is ultimately limited by the unconfined compressive strength of its material. It also decreases as the rise of the arch is reduced from an optimum profile toward a flat arch that can fail as a beam. Stable arches in unsupported mine roofs develop a rise that commonly approaches half the span width (Franklin, 1989), which is the profile inside an optimum circular voussoir arch. Any material that does remain beneath the compression arch within a rock mass is in tension, and its incidental loss increases arch stability by reducing the load on it. Buttress stability is critical to voussoir arch integrity, but normally offers no problem in a rock mass over a cave. Load on the arch is imposed by any material above the compression zone that is not self-supporting by its own voussoir arch. Distributed loads, from either a soil cover, a road base or a raft foundation require only a modest thickening of the arch to carry the increased stress in compression. Point loads, from column bases or other small foundation pads, threaten to fail an arch by its distortion and ultimate buckling, and are only safely carried where their stresses are distributed through an adequate

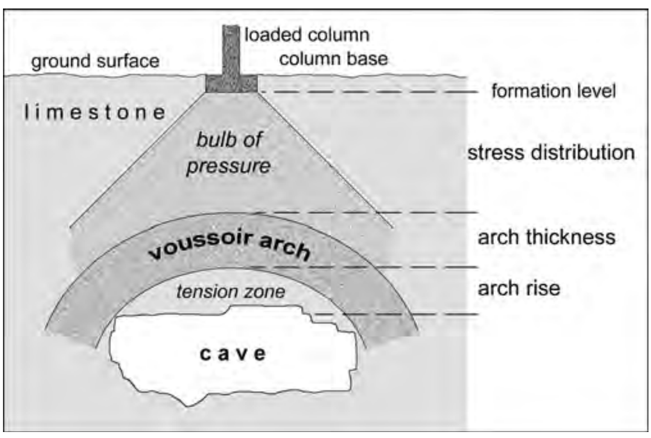

Figure 7.2. Elements of a voussoir arch developed in a fractured rock mass that spans a natural cave and carries additional load imposed by engineering works.

thickness of passive cover across the compression arch. If load distribution is assumed to spread beneath an angle of 45°, a zone with a depth that is about 0.7× the cave width is adequate to ensure no buckling of the voussoir arch, and less depth is sufficient where the foundation pad occupies a large proportion of the cave width.

The thickness required for stability in a rock roof over a cave is therefore the sum of the three components – arch thickness, arch rise and stress distribution zone (Figure 7.2). Structural analysis may determine safe values of these components for any cave width and for any applied load. Cave height is irrelevant as wall failure is not a threat in strong karstic limestone, and there are none of the thin pillars that are critical to the evaluation of mine stability. A cave with a steeply arched roof has no rock within the tension zone inside the arch rise, and also offers efficient load transfer to its wall rock; it may therefore be stable with a thinner roof over its centre-line.

However, such analysis is only with respect to the compressive strength of the rock mass, for which an overall value may be assigned. The additional hazards in a natural rock mass derive from the more variable factors. Fractures may fail in shear where they are orientated so that stress is imposed at highly oblique angles (unlike those imposed normal to the joints that are radial within an engineered dry stone arch). Wedge-shaped blocks can destroy arch integrity, though they are commonly held in place by confining stress and high friction angles on rock fractures. Fissures that have been partially and irregularly opened by dissolution, may carry no shear stress across their voids, but are likely to be locked by compression across areas of block contact. Thin beds may be overstressed to the point of buckling, but roof loads are generally carried by the thicker beds (where these are lacking, natural collapse should be widespread and recognisable). There is commonly no answer to these variations except to raise the factor of safety yet further, though inspection of accessible caves may reveal rock structures that should be treated as if they are either more or less safe than those in the ‘‘typical’’ rock mass.

7.2.2 Modelling the failure of cave roofs under load

Numerical analyses of loaded cave roofs suffer from the twin difficulties of modelling fracture patterns reliably and of applying realistic strength values for a complex rock mass. There is a shortage of published results.

Using the finite difference code Fast Lagrangian Analysis of Continua (FLAC), cave roofs have been modelled in 2-D at Nottingham University (Lu Zhengxin, pers. comm.). Caves, 3–50 m wide with flat roof profiles at depths of 2–10 m, were modelled under loads applied to pads of 1 m^2 at the ground surface above the centre-line of the caves. Load was increased until failure was defined by settlement of 25.4 mm, which indicates loss of integrity and is likely to precede total collapse, besides causing significant damage to built structures. The caves were modelled in materials of various rock mass rating, using packages of strength and deformation values that have evolved through research and appear to provide realistic results (Asef *et al.*, 2000). Results for values of RMR of 20–50 effectively show ultimate bearing pressures in terms of cave width, roof thickness and rock mass strength

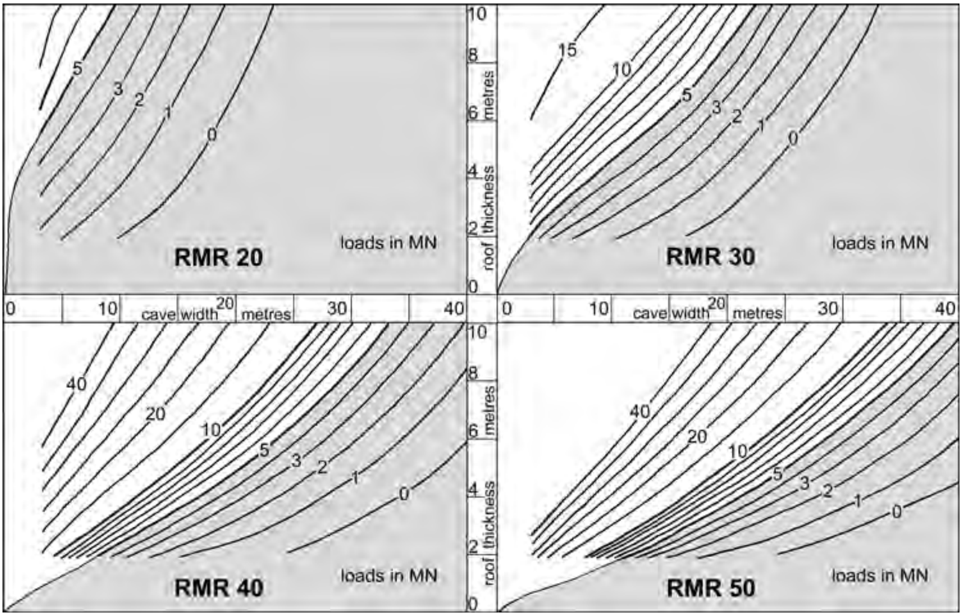

Figure 7.3. Nomograms that relate failure loads to cave width and roof thickness in ground of various rock mass ratings; loads are applied to foundation pads 1 m^2 on the surface directly above the caves; the shaded areas represent situations with respect to cave width and roof thickness where loading of 1 MN on the pad leaves a factor of safety <5, and are therefore considered unsafe.
After Lu Zhengxin, pers. comm.

(Figure 7.3). For any design load, selected factor of safety, estimated RMR and known or inferred cave width, a safe roof thickness can be determined from Figure 7.3. Continuation of the modelling showed that, in typical karst ground of RMR = 40, caves about 50 m wide collapse with no imposed load even at depths of 10 m, while caves 30 m wide collapse naturally when their roof is reduced to about 2.5 m thick. Where no cave exists, the same rock mass exhibits settlement of 25 mm at loads of 45 MN, indicating a safe bearing pressure of about 9 MPa.

Cave dimensions at failure loads of 5 MN can be extracted from these FLAC models to define safe conditions for the single case of 1 MN loading, on the 1-m^2 pad, with a factor of safety of 5 in any given rating of rock mass (Figure 7.4). If RMR for typical cavernous karst in strong limestone is taken conservatively as between 30 and 40, a cover ratio of the roof thickness being half the cave width ($t = w/2$) appears to be adequate for most engineering practice. In karst terrains on chalk and some other weak limestones, RMR may be estimated as nearer 20, and a cover ratio whereby roof thickness equals cave width ($t = w$) may be required for safe construction. Implications from this numerical modelling, with respect to cave roof integrity under imposed load (Figure 7.4), are only based on generalised estimates of the strength parameters for rock masses of the various rating values.

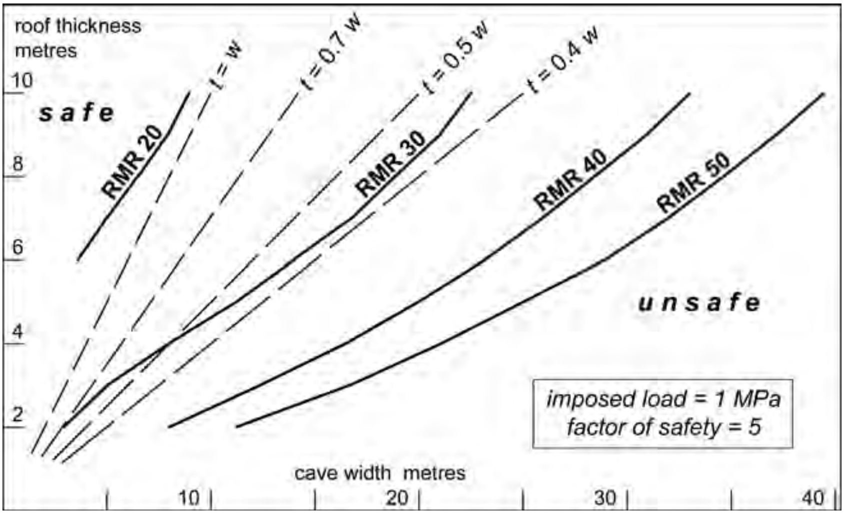

Figure 7.4. Envelopes of acceptability, with respect to cave width and roof thickness, where engineering loads of 1 MN are applied, with a factor of safety of 5, to pads of $1\,m^2$ on the surface directly over the caves in ground of various rock mass ratings; broken lines indicate various ratios of roof thickness (t) to cave width (w).

These estimates can only approximate the variable conditions in karst, where strong intact rock is broken by open or soil-filled fissures, in styles very different from those in insoluble rock masses. The data on caves could be improved by more specific modelling of fissured karst, with models designed to address the block mechanics of individual sites, and research is continuing with this aim. The current data is also derived from purely 2-D modelling, and failure loads are likely to be higher where some roof support is provided in the third dimension.

Finite difference analysis, also 2-D, of a cave roof in strong limestone in Tennessee did not incorporate structural loading, but did assess the effects of comparable seismic loading with a vertical peak acceleration of 0.14 g (Siegel *et al*., 2003). Results showed that a roof thickness equal to that of the cave width provided a factor of safety of 2.5–3.0, which reduced to 1.5–2.0 where the thickness was half that of the width (Figure 7.5). This stability was under a dynamic seismic loading that was equivalent to little more than normal highway loading. When compared to practice from elsewhere, these factors of safety appear to be low, but probing to a depth of 3 m was regarded as adequate for the local site conditions where caves are typically up to 6 m across.

Numerical and physical modelling of artificial caves beneath loaded column bases has provided data on the stability of weak, massive sandstone that has a rock mass rating of 30–40, rather lower than that of typically stronger, but more fissured, karst limestone (Waltham and Swift, 2004). In the homogeneous sandstone, plug failure was the main cause of structural collapse over the artificial caves. A roof thickness that was half the cave width ($t = w/2$) was shown to be stable under the

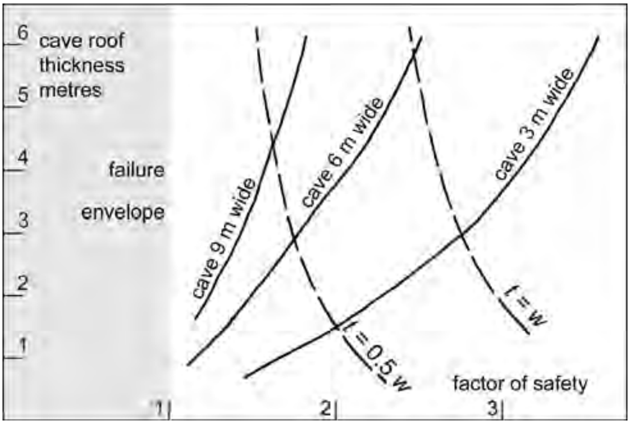

Figure 7.5. Numerical modelling of roof failure in strong limestone over caves of various widths under modest seismic loading in Tennessee; ratios of roof thickness (t) to cave width (w) are also indicated.
After Siegel *et al.* (2003).

maximum loads permitted by local building regulations. This implied that such a ratio was appropriate and perhaps conservative over typical limestone caves. Perhaps more significant than the absolute results, comparative data showed the importance of load position over the cave. Even where only a proportion of the column base footprint is over solid ground beside the cave, the bearing capacity is greatly increased. This shows that caves obliquely below structural foundations do not present a hazard; probes that are splayed out from the corners of a structural footprint appear to be unwarranted in the proving of solid ground. A full-scale loading test of a cave roof over the same sandstone was used to validate the modelling data. Failure of a roof 0.5 m thick, over a cave 4 m wide, occurred under a load of 340 kN applied to a pad 400 mm square, creating a plug through a zone of oblique stress fractures (Figure 7.6). The results from that rare opportunity of a real loading test do conform with the interpreted and calculated values for cave roof stability elsewhere.

7.2.3 Punching failure of cave roofs

A punching failure may develop where a thin slab of rock fails in shear round the perimeter of a small loading area – such as a pile tip or a small column base – so that a plug of rock is displaced into an underlying void (Figure 7.7(a)). Resistance to failure is therefore a function of the shear strength of the rock mass and the wall area of the plug. Shear strength is immensely variable for a rock mass, as it depends on the immediate disposition of fractures and fissures; a very approximate figure may be taken as about one-tenth of the shear strength of the intact rock, therefore about 3 MPa for strong karstic limestones. Plug wall area increases with rock depth (roof thickness), but part of the depth is lost in the development of a flare on the lower part

Figure 7.6. Broken rock in a cave roof loaded to failure; a full-scale test (by loading upwards on hydraulic jacks) on homogeneous sandstone in Nottingham, U.K.
TW.

of the plug, where it therefore fails in tension at lower unit values; this is a normal failure mechanism in homogeneous rock, but is distorted by pre-existing fractures within a rock mass (Figure 7.7(b)). Plug wall area also increases in proportion to the perimeter of the loaded pad. A square foundation pad quadrupled in area only has its perimeter doubled, so can accept only half the loading stress where plug failure may occur; guideline values for safe bearing pressures are less relevant than total loads in this situation.

Under these conditions, safe loading of a cave roof may be defined theoretically by roof thickness and pad size (Figure 7.7(c)); these calculated bearing capacities incorporate a safety factor of 5, and any lower factor would be inappropriate where unknowns remain with respect to the behaviour of a fractured and fissured rock mass. The implication is that rock proven to conventional limits of around 3 m will not develop a plug failure over a cave. Risk is eliminated by using larger foundation pads, but a small inherent risk can remain with respect to pile tips.

A plug failure could conceivably develop under lower loads where a roof block, bounded by pre-existing fractures through the entire roof thickness, is punched through by a point load imposed exactly over it. The chance of this happening is remote. In reality, structural loading over a cave generally closes the rock fissures and thereby develops a stronger voussoir arch within the roof mass. Punching failures have been recorded in the weak Florida limestones (Sowers, 1975) but only where excessive loads were placed on crusts of limestone just 1–2 m thick over unconsolidated sand. The sand's presence was known, but had been thought to be stronger; conventional proving of the ground should reveal any caves at such shallow depths beneath planned foundations.

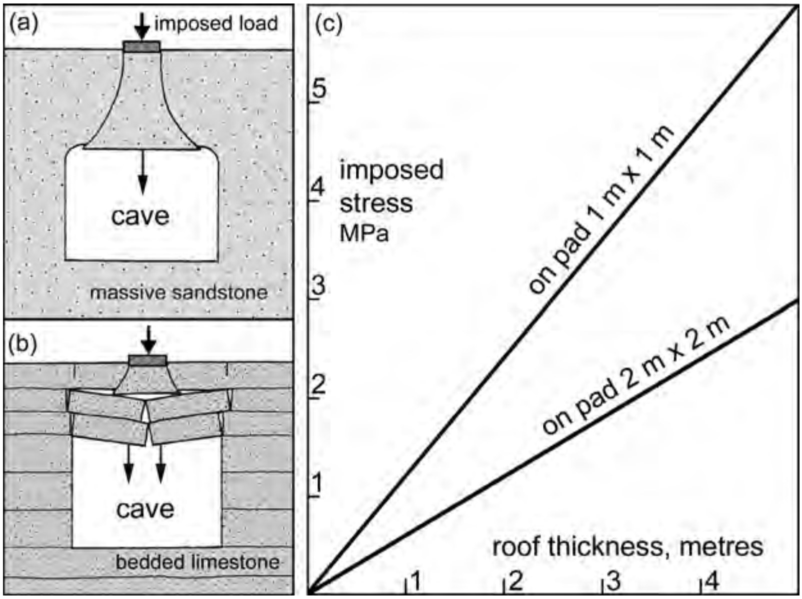

Figure 7.7. Plug failures over caves. (a) Typical plug profile in homogeneous rock. (b) Typical complex failure in bedded and fractured rock. (c) Bearing capacity on limestone with respect to plug failure over a cavity, in terms of the size of foundation pad and the thickness of sound roof rock; the values have a safety factor of 5, and the rock mass has a shear strength of $3\,MN/m^2$.

7.3 SAFE COVER THICKNESS OVER CAVES

All the available information suggests that the "rule-of-thumb" that cover thickness should exceed cave width is excessively over-conservative in most of the strong limestones that form cavernous karst. Evidence from the various available sources suggests that a roof thickness of about half the cave width is stable and safe under most conditions of loading. In view of the extreme variability of karstic ground conditions, a guideline that roof thickness should exceed 70% of cave width (i.e., thickness/width = $t/w = 0.7$) is probably more appropriate in most karst terrains in strong limestone. This value is still conservative under normal structural loading, and is very conservative under highway loading. Guideline values of safe cover thickness must be modified to accommodate cavernous rocks of varying strength, notably increasing in gypsum, chalk and weak limestones, while decreasing in strong basalt (Table 7.1). Within this table, imposed loads are maxima that are 50% of the Safe Bearing Pressures applied to sound rock; quoted safe roof thicknesses are of sound rock, ignoring any soil cover, or any epikarst zone of weathered, fissured or pinnacled bedrock.

At individual sites, specific site details, local knowledge and local experience may dictate safe roof thicknesses (and therefore probing requirements) that are higher or

Table 7.1. Safe roof thicknesses for various cave situations – and therefore a guideline to appropriate depths for probing to prove sound rock prior to construction.

Rock	Imposed load	Karst class	Cave width – likely maximum	Safe roof thickness
Strong karstic limestone	2,000 kPa	kI–kIII	5 m	3 m
		kIV	5–10 m	5 m
		kV	>10 m	7 m
Weak limestone and chalk	750 kPa		5 m	5 m
Gypsum	500 kPa		5 m	5 m
Basalt lava	2,000 kPa		5–10 m	3 m

lower than the tabled values. Site inspection may reveal limestone that is significantly more or less fractured and fissured than is typical. In local zones of heavy rock fracturing, accurate figures for safety factors cannot be achieved, and only broad guideline concepts can be applied. Experience in rock tunnelling suggests a variable cover ratio (Sowers, 1996), where sound rock requires a cover of half cave width while more fractured rock requires cover that exceeds cave width (though apparent typing errors inverted this relationship on page 157 of that excellent book). Long-term decline in rock strength, due to fracture propagation where it is under stress (Tharp, 1995), has implications for the stability of individual rock beams; however its impact on a compression arch within fractured rock would appear to be minimal, and is adequately covered by the very high safety factors incorporated into concepts of rock mass strength.

An alternative approach to the safe cover thickness is based on the decline of imposed stress at increasing depths beneath a loaded foundation structure. It has been suggested that induced collapse of a cave roof is unlikely where the loading stress is less than 5–10% of the existing overburden stress (Sowers, 1996). Reference to the undistorted bulbs of pressure perceived by foundation engineers suggest that this stress ratio is reached at a depth of about 4 m beneath a foundation pad 1 m^2 carrying a load of 1 MN, where overburden stress increases by 25 kPa per metre depth. This takes no account of cave width, and assumes there is no cave roof at a critical state of imminent collapse. It is however slightly conservative because it does not account for stress redistribution around an open cave, where wall failure is unlikely. A safe thickness of 4 m is commensurate with guideline figures derived from other considerations. Where a foundation pad 2 m^2 carries a load of 4 MN, still with an applied stress of 1 MPa, the imposed stress exceeds 10% of overburden stress at a depth of about 6 m. This implies that greater thicknesses of sound rock should be proven where heavy structural loads are placed on karstic rock that may contain large caves. There are multiple benefits in using larger footings that impose lower stresses on cavernous ground.

Any accessible cave may be inspected so that its roof stability is assessed at least semi-quantitatively, and reasonable precautions are then be taken with respect to either remedial works or reduction of imposed loads. The obvious requirements for

Table 7.2. Some of the various guidelines that are recommended or applied during ground investigation to prove that limestone is free of voids by probing or drilling beneath planned foundations.

Source	Location	Probe depth	For loading	Cave width	Rock type
City of Rochester	New York	1.5 m	< 4.8 MPa	n.s.	Strong dolomite
City of Allentown	Pennsylvania	1.5 m, or 2 × pd	implied	n.s.	Strong limestone
Foose *et al.* (1979)	Pennsylvania	2.5 m	n.s.	n.s.	Strong limestone
Cooley (2002)	Alabama	2.0–3.0 m	0.9-m piles	n.s.	Dolomite, 70 MPa
Sowers (1996)	Florida	1.5–3.0 × pd (1.5–6.0 m)	implied	n.s.	Limestone
Knott *et al.* (1993)	Pennsylvania	3.0 m	1.0 MPa	n.s.	Dolomite
Wyllie (1999)	n.s.	3.0 m	n.s.	n.s.	Limestone
Raghu (1987)	N.E. U.S.A.	3.0 m	n.s.	< total pile cap	Strong limestone
Erwin and Brown (1988)	North Carolina	3.0 m*	< 1.2 MPa	n.s.	Weak limestone
Waltham *et al.* (2003)	Karst kI–kIII	3.5 m	< 2.0 MPa	< 5 m	Stong limestone
Higginbottom (1966)	U.K.	3.6 m	high-rise	n.s.	Weak chalk
Sotiropoulos *et al.* (1979)	Greece	4.0 m	heavy pile	n.s.	Limestone
Wagener and Day (1986)	South Africa	4.0 m	n.s.	n.s.	Strong dolomite
Tan and Batchelor (1981)	Malaysia	4.5 m	high-rise	may be large	Strong limestone
Garlanger (1991)	Florida	4.9 m	n.s.	n.s.	Limestone, 30 MPa
Tan (1987)	Malaysia	5.0 m	high-rise	may be large	Strong limestone

* Specified as 1.5 m of sound rock and the next 1.5 m below with no void >150 mm.
n.s. = not specified. kIII = karst class. pd = pile diameter.
Strong limestone = typical karstic limestone with UCS > 70 MPa.

safety, and natural desires to err on the side of caution, mean that most caves with rock cover less than their width will be filled with concrete where they lie beneath a construction site. The problem lies not with the known caves, but with the unknown caves. Many of these can only be found by protracted and expensive programmes of probing, with or without geophysical surveys. A pragmatic approach on cavernous ground can be to omit the costs of potentially inconclusive ground investigations and spend the savings on improving any new construction so that it will not cause or suffer from ground collapse. Buoyant foundations and spread footings on rafts or beams can reduce imposed loads so that rock collapse is not induced in ground that has been stable when undisturbed through geological time. Piled foundations can support structures designed so that any single pile failure does not destroy integrity. In the words of one practical engineer, "you can buy a lot of reinforcing steel for the cost of a geophysical survey, and then not worry about the chance of caves being there".

Without taking the pragmatic approach in its entirety, some guideline value of safe cover is required to plan any ground investigation by probing, and a variety of numbers is available from published sources (Table 7.2). Some of these variations are in respect of local available data on the maximum and typical widths of caves that can be anticipated. It would appear that probing depths of <2 m are inadequate except where there is a substantial databank confirming that only very small caves are likely to occur. It is also significant that the only two probing depths greater than 4 m are one on a weak limestone and one in a karst terrain notable for its large caves. Heavy-duty, cast-in-place, end-bearing piles may require 4–5 m of proven sound rock in any environment. However, large-diameter concrete piles carrying loads up to 10 MN have been placed on the weak limestones of Florida simply on the basis of load testing of each pile, without any specification for probing beneath the pile tips.

It is notable that guideline values of safe cover, cited above or in Table 7.2, bear no relationship to the figures widely used in assessing stability over old mine workings. A safe cover of 10× mine height relates only to the hazard of void migration (to form crown holes) in thinly bedded Coal Measure rocks; it is meaningless with respect to cave roofs in karst limestone.

7.4 EXISTING STRUCTURES OVER CAVES

Numerous built structures have been placed over open caves that lie at shallow depths, which were either known or unknown at the time of construction. Some were built after due remedial works had been completed. Others were built on natural ground. A few of these have failed (Section 7.1), but the remainder survive – some in conditions that may be described as marginal.

Natural Bridge, in Virginia, is effectively a very short cave that is about 27 m wide with a roof thickness of 14 m ($t/w = 0.5$) of thickly bedded strong limestone (Figure 7.8). It has carried a two-lane highway for a hundred years and shows no sign of distress. A main road in Iceland crosses over the lava tube of Raufarholshellir where it is about 10 m wide under a basalt roof just 4 m thick ($t/w = 0.4$). When the cave was found (from a nearby entrance) long after the road was built, concerns led to checking it for movement and roof breakdown, but none was found over some years of monitoring, and the cave and road are now regarded as stable. In Hungary, the urban sprawl of Budapest includes roads and buildings that stand over some 30 km of known cave, much of which lies at shallow depths and where no collapse has ever been recorded.

In the karst of western Ireland, the railway from Dublin to Sligo stood for many years on limestone only 2.5 m thick over a passage more than 6 m wide ($t/w = 0.4$) in the St. Augustine's Cave. When the single track railway was replaced by a main road around 1960, a ground-slab of reinforced concrete effectively carried the road over the cave in order to minimise any risk of collapse, and a length of masonry walling was built inside the cave to reduce its widest section of unsupported span (Figure 12.13). A major building in Huntsville, Alabama, lies over a cave 12 m wide under about 5 m of rock ($t/w = 0.4$), but is supported on piles that pass beside or through the cave (Case study #6).

Bowling Green, Kentucky, has many roads and buildings that stand over the large cave passages of the Lost River System. Many stand, in apparent stability, on rock spans with t/w ratios that are around 0.5, though the Dishman Lane collapse was inevitable where it stood on a cave roof with $t/w = 0.1$ (Case study #2). On the other side of town, a four-lane highway passes directly over a chamber in Bypass Cave, where a flat roof nearly 10 m wide has only about 2.5 m of rock and 2.5 m of soil cover between itself and the road. With a ratio of $t/w = 0.25$, this lies outside the perceived wisdom for safe engineering, even with the low imposed load from only a

Figure 7.8. Natural Bridge, Virginia, with a road over its thin but stable arch.
TW.

highway. The continuing survival of the road suggests that guidelines on safe cover ratios are comfortably conservative (unless the road is in fact dangerously close to collapse).

An open cave was discovered under the main runway of Palermo airport, in Sicily, Italy, during adjacent routine maintenance works (Jappelli and Liguori, 1979). The critical part of the cave was a single chamber 40 m long and 30 m wide with an uneven but roughly horizontal roof draped with stalactites (Figure 7.9). This was separated from the base-course of the runway by just 2–4 m of limestone, and the upper part of that was described as brecciated due to weathering. With a cover ratio (t/w) of 0.1 or less, the risk of collapse was deemed unacceptable, even under only the distributed load of the reinforced concrete runway; the entire cave was filled with concrete (injected through a grid of boreholes on 5 m centres). This cave could be regarded as having been a close call on the runway's safety. On the other hand, it

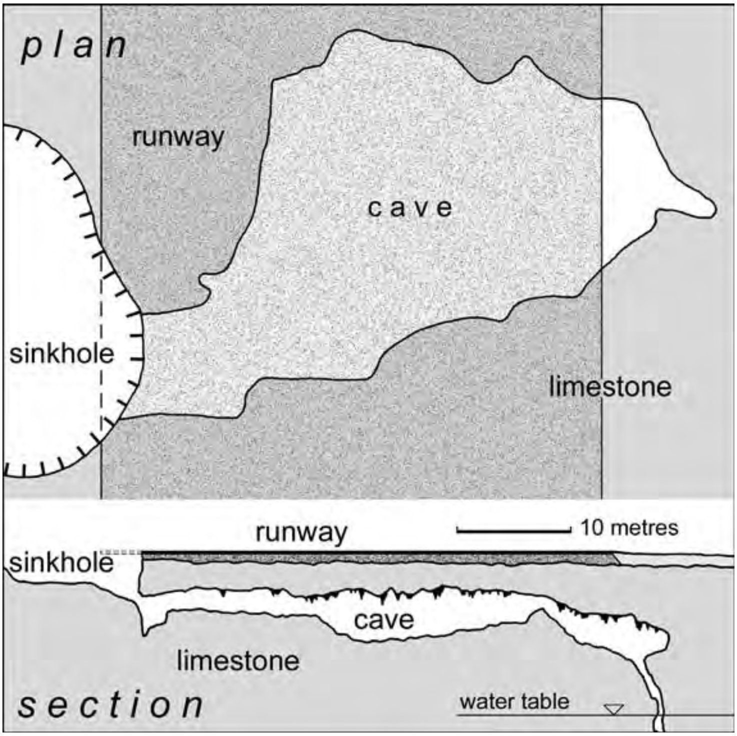

Figure 7.9. Plan and section of the cave found beneath the runway at Palermo airport in Sicily; on the section, part of the limestone immediately beneath the main cave and the sinkhole floor is a layer of breakdown blocks of unknown depth.
After Jappelli and Liguori (1979).

may be viewed as an indicator of the stability of a very thin span of fissured rock over a natural cave. It may indicate the importance of spread loading, by the runway structure, and it was perhaps surprising that the cave did not collapse under the disturbance of construction traffic. Overall, it does appear that guidelines demanding rock cover at least half as thick as a likely cave span ($t/w = >0.5$) appear to be adequate and appropriate in engineering works on most cavernous ground.

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