INFLUENCE OF PACK DESIGN UPON GATEROAD DEFORMATION IN SOFT FLOOR CONDITIONS

By
A.K. Isaac¹ and A.R. Payne²

ABSTRACT

The high cost of gateroad repairs and maintenance in U.K. longwall coalmining is a clear indication of the failure of permanent support systems to fulfill their prime function, i.e. control of strata movement. The main component of gateroad support is the pack, the design and materials of which have experienced an important development process during the last 10-15 years. More recently, research investigators have attempted to give a design basis to pack quality and dimension, and reference is made to two theoretical approaches.

In the course of a current research programme based upon in situ monitoring, interim design recommendations have been proposed as a means of affecting improvement in support efficiency. Identification of characteristic deformation behaviour forms the basis of the recommendations made and the subsequent improvement in gateroad performance reported.

INTRODUCTION

Permanent gateroad support design at longwall faces has developed with technological advance and changing geological environments. Coalface mechanisation and new support materials introduced into deeper and often more complex strata, have stimulated the development of technically feasible and cost-effective systems designed to promote high output and gateroad stability.

The major element of gateroad support is the pack which aims to generate stability for short and long term. Hence, it requires the ability to resist initial loading produced by face end extraction, and to develop adequate resistance as it consolidates under long term loading with face movement away. This ability to develop acceptable load/deformation behaviour is governed by the nature of proximate strata, particularly that of the floor foundation. Roof strata also require to be reasonably competent to maintain the necessary beam-like behaviour.

For the achievement of higher outputs and improved control, hand and mechanical packing methods have since 1970, been gradually superseded by a number of alternative methods. Although based on the principle of hydraulic emplacement of monolithic materials, flexibility of design has witnessed, for example, a composite pack employing components that satisfy short and long term requirements.

An important requirement in the development of new packing systems has been the compatibility to be achieved between pack "stiffness", i.e. its load/deformation behaviour, and floor strength. Methods have been developed for assessment of pack loading, and physical property evaluation has been conducted using traditional laboratory techniques. Instrumentation has been applied for in situ monitoring to establish characteristic deformation behaviour of the gateroad. From such data, attempts are currently being made to formulate predictive techniques for support design.

The major difficulty facing designers in soft floor conditions is the non-linear behaviour of the support foundation. The ability to limit the time-dependent movement of this foundation is a step forward in establishing interim and long term solutions to the problem. The paper describes case studies of three designs of permanent roadway support where strata control in soft floor conditions has been relatively successful.

DEVELOPMENT IN PACKING SYSTEMS

DESIGN CONSIDERATIONS

The main support functions of a roadside pack are:
- to give support to roof strata bridging the roadway, so generating uniform load distribution;
- to effect waste control near the face end.

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to satisfy these requirements, packs need to fill completely the cavity between roof and floor and be erected in phase with face advance. This provides early support to the immediate roof beds and long term support to the upper beds.

The traditional labour intensive and generally inefficient hand-built stone packs have increasingly given way to alternatives regarded as elements of an integrated roadway support system. The adoption of monolithic pump packing systems since 1973 has been accompanied by research investigations intended to produce a basis of design related to pack loading.

The Detached Block Theory (Wilson, 1975; Whittaker et al, 1977) assumes that waste caving governs caving height. Accordingly the roadway support system controls the block of strata below the upper beds (Fig. 1(a)). This approach appears satisfactory for the face support role and has been incorporated into the U.K. National Coal Board Code of Practice (Production Instruction 1982/6). However, it does not take account of the load imposed by the bridging beds, and roadway deformation behaviour behind the face line provides evidence of the developing nature of pack load.

The Roof Beam Tilt Theory (Smart et al, 1982) considers the bridging beds to generate the load as the pack compacts under the downward movement of the immediate roof. The roof tilts from ribside towards the face, the amount of tilt being controlled by pack and foundation strength (Fig. 1(b)). Pack load may be calculated from pack stiffness value and roof bed tilt, both of which have been incorporated into appropriate equations. This approach aims to limit the roof bed tilt to a maximum of 2.5° in order to prevent ribside roof fracturing, and to ensure that bearing strength of floor strata is not exceeded. This analysis was developed from investigations into long term stability of gate roads (Isaac et al, 1983).

METHODS OF PACK CONSTRUCTION

The four main methods of building packs are:

- manual
- mechanical
- pneumatic
- hydromechanical

Hand-built packs utilise ripping of caved debris, hardwood or softwood, or wood substitutes e.g. Autoclaved Aerated Concrete blocks and wiremesh reinforcement. The most common material used until recently has been stone because of its convenience and comparatively low cost.

The failure of stone packs to control roadway deformation stimulated the development of packs comprising other materials such as wood which provided improved early load bearing qualities. However, importation of the larger amounts of wood required, the high cost involved, and the relatively low stiffness of wood blocks, generated further interest in alternative systems of pack construction.

Mechanised packing systems employing slushers, blades and rams (Evans & Trolley, 1974) have found a gradual but steady increase in application because of their technical benefits compared with manual systems. In most situations, mechanically built packs improved the rate of pack construction and reduced the number of personnel involved. However, there being no roadside wall sometimes caused a weakening of the roadway shoulder which in turn tended to produce instability and support distortion.

Pneumatic stowing of graded ripping material has been performed successfully with high pressure or low pressure units. High pressure systems with compressed air at 0.4-0.6 MPa, can generate quality packs with material up to 0.25m³ size. Such packs can be further reinforced with cement, resin, or naturally cementitious materials. Although capital costs are higher, low pressure stowers are sound-proofed, operate at pressures of only 0.08 MPa, and are specifically designed for pack hole filling. A high quality pack can be constructed from 8cm material, but dust is a problem in both systems.

Hydromechanical packing systems form the bulk of monolithic packing methods (Clark and
Newson, 1985). The method derives in part from the routine use of tailings or sand slurry emplacement in metal mines, and also as a technically superior system compared with other methods. The general principle of pump packaging systems entails the mixing of an active ingredient with water to form a slurry which is pumped to the pack hole where it mixes with a further slurry mixture containing agents for ease of flow and acceleration of setting.

The methods now generally referred to as monolithic pump packaging systems were first applied at Brynlliw Colliery, South Wales, 1973 (Thyssen System), at Rm Heath Colliery, Staffordshire, 1979 (Aquapack), and at Littleton Colliery, Staffordshire, 1982 (Tekpak). Recent developments include the use of synthetic anhydrite and fly ash (Flashpack), both of which offer assistance in special cases.

**MONOLITHIC PUMP PACKING SYSTEMS**

Application in the U.K. coalfields of the available systems has been previously described (Hodgkinson, 1977; Collier, 1984; Mifflin, 1985).

The technique may clearly be seen as one of continuous development in terms of materials, equipment and application. The original Thyssen System was a major technological innovation in an important area of coal face and gateroad observation. The use of slurries in pack construction at coal mines represented a major departure from traditional methods and a continuing process of refinement has ensued. Appendix I gives the main technical characteristics (Johnson et al, 1982) of the three systems, the cementing agent in the mixtures being inferred from their trade names, i.e. Packbind, Aquacem and Tekcem. The elimination of ROM coal aggregate in the latter two systems produced an essentially cement based slurry with the high water contents shown. The other satisfactory parameters of pumping life, setting time and compressive strength, imply that future pump packing systems will move in the direction of Tekpak. A recent development in this system is the use of Tekpak XX with its pumping life of 20-24 hours which requires less flushing out of pipelines and an important consequence, less water on gateroad floors. Setting times and compressive strengths are the same as Tekpak.

**SITE LOCATION**

Bettes Colliery, located on the north west rim of the South Wales Coalfield, was developed to extract anthracite coal from the 1.4m thick Red Vein in the Middle Coal Measures. Mine design included the exclusive use of retreat longwall coal faces with dual life rectangular gateroads to maximise extraction of the available reserves (Fig. 2). Comprehensive investigations have, since 1978, been conducted in the West District gateroads in order to optimise gate and permanent support design. At WI and W2 Districts, the proximate strata consisted of a medium strength laminated silty mudstone roof, with a medium strength silty mudstone underclay floor that blended into mudstones with depth. However, as extraction progressed westwards with W3, W4 and W5 Districts, floor strength in the lower sections of the gateroads decreased. The occurrence of weak coal with associated soft underclay and increasingly weaker mudstone floor strata, produced a highly stratified floor of weak beds 0.75-1.0m thick with plastic interfaces. Localised washouts and rolls in the immediate seam roof coincided with the zone of weak floor to produce an area of greater instability than had previously been the case at Betts.

**PACK DESIGN**

Initial trials at the neighbouring Abernant Colliery, and at WI Gateroad, Betts, led to the development of a multi element pack support system. Wood chocks, comprised of hardwood blocks, were used to provide immediate support behind the face and powered supports. The Thyssen pump pack system was employed to fill the chocks and spaces between to provide long term high load support. Thus the 'Standard Betts' pack (Fig. 3(a)) comprised 5 elements, labelled from the wasteside as:

1. Wasteside unfilled wood chock
2. Wasteside pump pack
3. Centre filled wood chock
4. Roadside pump pack
5. Roadside filled wood chock

Elements 1-4 were located on the seam floor, while element 5 was sited approximately 0.5m lower on the gateroad floor. A chock was included to provide lateral support to the ribside while the gateroad maintained its development support system of 5.0m R&J roof bars.

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on wooden props.

Successful re-use of W1/W2 and W2/W3 Gateroads with minimal repairs in the medium strength floor led to the use of the standard support system in W3/W4 Gateroad. However, with the weaker floor, severe gateroad deformation occurred and considerable floor heave. The first modification in the lower section of W4/W5 Gateroad was the introduction of a narrower 3 element pack, and an arcade designed to absorb floor heave from the gateroad. Continuing pack instability led to the second modification with the reintroduction of the 'standard system' set on a raft foundation of 2 x 8 R2J bars, under each row of chocks.

RESULTS

Fully instrumented investigation sites were installed in the 'standard' Betws support system of W3/W4 Gateroad (Site B.W.3.1), and in the raft foundation zone of W4/W5 Gateroad (Site B.W.4.2). A satellite site (Site B.W.4.1), was also placed in W4/W5 Gateroad. Vertical closure was monitored with convergence recorders, and roof and floor strata closure and bed separation with 1m and 2m strata bolts. Pack element loads were monitored with hydraulic cells (flat jacks) and pack struts provided pack compaction. Coal seam lateral deformation remote from the gateroad was measured with a magnetic multi reference point extensometer, and a 3-dimensional multi reference point


(a) Gateroad Permanent Support Systems

25m W3 Face Advance

Support Load (MPa)

0 2 4 6 8 10 12 14 16 18 20

0 2 4 6 8 10

500m W3 Face Advance

(b) Support Load Development

25m W4 Face Advance

Support Load (MPa)

0 2 4 6 8 10 12 14 16 18 20

0 2 4 6 8 10

500m W4 Face Advance

(c) Proposed Characteristic Deformation Behaviour

Figure 3 - Gateroad Support, Loads and Deformation, Betws

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displacement probe was used for floor strata deformation. The instrumented sites installed in W1/W2 and W2/W3 Gateroads provided comparisons with the results obtained from soft floor conditions. Permanent support surveys conducted along gateroad length, revealed height, width and roof inclination changes, and roof, pack and floor deformation. These representative surveys were conducted to substantiate results from investigation sites, and recorded the influence of geological or support changes on gateroad deformation.

The effect of soft floor conditions was that of increased gateroad deformation at Site B.W.3.1 ahead of W3 Face. Lateral extrusion of the coal seam and seatearth occurred 125m ahead of the face, caused roof beam tilt towards the face side, and increased the rate of vertical closure with the influence of the front abutment zone. At the face, roof beam tilt was 3°20', with a 4.5m yield zone, which compared with 2.3m for W1/W2 Gateroad.

The support loading profile (Fig. 3(b)) showed that the initial load being the stiffest element, up to a limit of 1.5 MPa, compared with 10.0 MPa at W1 Gateroad, at which a reduction to 0.3 MPa occurred, suggesting possible floor failure under the element. Excess load was transferred to the neighbouring pump pack elements, 2 and 4, which subsequently rose to 1.0 MPa. Lateral extrusion of the failed seatearth from under the pack pushed element 5 into the gateroad, decreased pack stability and increased roof beam tilt to an excessive 9°20' with a potential ribside break. Increased loading on the ribside chock to 1.7 MPa resulted from this situation. Total vertical closure of over 50% was produced at an increased rate of 1.22mm/day compared with 0.22mm/day in the harder floor conditions at W1/W2 Gateroad. Pack and ribside extrusion caused 30° lateral closure. Due to severe gateroad deformation as shown in the characteristic deformation behaviour (Fig. 3(c)), Site B.W.3.1 became inaccessible and a 0.6m dint along the entire gateroad length with removal of the roadside chock, element 5, was required before its re-use by W4, the second face.

In order to control the amount of floor heave expected in W4/W5 Gateroad, the first pack modification was introduced at Site B.W.4.1. As at Site B.W.3.1, the effect of the front abutment occurred well in advance of the face, at 150m, with commencement of roof beam tilt. Instability of the pack caused by failure of the soft floor and extrusion of the roadside chock into the gateroad produced a high final roof beam tilt of 4°. Vertical closure was reduced with only 72% on the packside and 18% on the ribside, which may indicate that a higher stress was imparted by the soft floor and resulting ribs more than compared with the unstable pack, producing greater floor heave. Repairs required an 0.8m dint plus removal and replacement of roadside and ribside chocks for re-use of this section of the gateroad by W5 Face, i.e. the second face.

At Site B.W.4.2, with the 'standard Betws' support system placed on the RSJ raft, pack stability was achieved with distribution of load over the full pack width as well as restriction of roadside chock extrusion. As at Site B.W.3.1, the stiffest element 3, accepted load to a maximum of 3.7 MPa (Fig. 3(b)) with surrounding elements taking an average of 2 MPa but without floor failure. Element 5 located on the gateroad floor, off the RSJ raft, did in fact cause floor failure, reaching a maximum of 1.5 MPa before reducing to 0.6 MPa and transferring the load to element 4. Lateral restraint produced by the RSJ bar also reduced extrusion of the roadside chocks and increased pack load and stability.

Increased pack stability reduced final roof beam tilt to 1°10' similar to that experienced in W1/W2 Gateroad in medium strength floor. However, vertical closure, from floor heave, substantially increased with 45% vertical closure on the packside and 20% on the ribside. This reversed behaviour from Site B.W.4.1 indicates that the higher stress on the soft floor was now imparted by the pack which had penetrated the floor. However, due to pack stability, only a floor dint of 0.6-1.5m was required at completion of W4 Face operations for gateroad re-use by W5 Face.

Both visual observations at dints and analysis of 3-D rock displacement probe results at Site B.W.4.2 indicated floor strata deformation under the pack, gateroad and ribside. Movement in the floor strata was detected as much as 120m in advance of the face to a depth of 7m with lateral strata movement into the gateroad, but negligible upward movement. Differential lateral extrusion of the soft floor strata at depths of 0.4m and 1.5m below the gateroad made the probe boresholes inaccessible immediately behind the face. Characteristic deformation behaviour proposed for Site B.W.4.1 (Fig. 3(c)) shows lateral extrusion of the immediate floor strata, which produced pack instability and caused floor strata extrusion. High stresses on the ribside caused penetration of the floor and increased floor heave on that side. However, floor failure is proposed to have been limited to the immediate 2m of floor strata. Increased pack stability at Site B.W.4.2 (Fig. 3(c)) increased the stresses transferred to the floor and restricted lateral movement. Hence the pack penetrated the floor, with increased floor heave and floor failure to greater depths.

COAL PILLAR PACK

SITE LOCATION

Coal pillar packs were used at Taff Merthyr Colliery for the protection of dual life gateroads between retreating longwall faces. Initial extraction of the Seven Feet Seam was by
conventional advancing longwall faces with arched gateroads separated by wide pillars, but to increase production, retreat faces with dual life gateroads were adopted.

B20 Face (Fig. 4) retreated downhill, was followed by the B21 retreating in the same direction, the intervening gateroad being used as a tailgate for both faces.

Proximate geology comprised a medium strength mudstone roof interbedded with thin bands of siltstone, and a 1m thick strong mudstone floor underlain by a very weak 1.5m mudstone followed by silty mudstones interbedded with thin carbonaceous mudstones. Floor strata under the seam was highly fractured and fissured.

Results

A fully instrumented investigation site was installed in the B20/B21 Gateroad (Site T.B.1.1) complete with magnetic reference point extensometers in both pillar and ribsides, and a 3-dimensional rock displacement probe in the floor. Full gateroad permanent support surveys were also conducted periodically.

The first indication of the front abutment effect of B20 Face as it approached Site T.B.1.1 was the development of yield zones on both sides of the gateroad, and roof beam tilt approximately 80m ahead of the face. Such effects were similar to those experienced in the soft floor conditions in Bcate 8/3/81 and 8/4/81 Gateroads. Loads (Fig. 5b) on the gateroad also started to gradually increase from 0.85 MPa. The rate of movement increased approximately 12m ahead of the face, after which gateroad closure and support loads rapidly increased.

Load on the arcade chock reached a maximum of 1.5 MPa after B20 Face had retreated 40m past Site T.B.1.1, while the pump pack achieved 1.8 MPa after 52m and the gateroad chocks 0.5 MPa. However, all subsequently reduced, viz. the pump pack to 0.9 MPa, arcade chock 1.0 MPa and gateroad chocks to 0.20-0.25 MPa, indicating general floor failure. These results show the improved floor load distribution characteristics of monolithic pack materials rather than point load of the wooden chocks.

Unfortunately, loads on the narrow pillar were not ascertained, but the yield in the ribsides yield zone depth of 2.3m, implied that it was higher than those accepted by multielement packs.

As the pillar was formed in a yield zone, it had already technically failed. Pillar extrusion was at its maximum immediately after passage of B20 Face (4.3m/m advance) as the pillar yielded, but this gradually reduced (0.6m/m) as lateral loads on the pump pack increased to a maximum of 1.1 MPa. Yield allowed a rapid increase in roof beam tilt to stabilise at a total of 5° 15' after 150m, confirmed by roof strata bolt movement.

Whereas roof beam integrity was maintained, considerable floor heave occurred during first face (B20) operations, producing 17% of a total, vertical closure of 25%. The majority occurred on the ribsides where large fractures parallel to the gateroad occurred in the thin hard seam coal. Lateral extrusion of the soft floor strata into the gateroad from under the ribsides and coal pillar caused buckling and failure of the upper hard layer. The 3-D rock probe showed that such movement was largely restricted to the top 3.5m of floor strata. Increased lateral sub-floor strata movement on the higher stressed ribsides produced increased buckling and floor heave on that side.

An 0.9m dink taken along the entire length of the gateroad, to prepare for the second face (B21) operations, confirmed floor failure under the gateroad supports which had
in fact penetrated the hard seatearth into the soft sub-floor strata, increasing the amount of floor heave.

Proposed characteristic deformation behaviour (Fig. 5(c)) indicated that floor strata movement was probably restricted to the immediate floor with failure under the ribside and pack, inducing lateral movement into the gateroad, producing the floor heave. Coal pillar stability increased by application of lateral restraint, maintained roof beam integrity and minimised extrusion.

SITE LOCATION

Monolithic Aquapack was first used in the South Wales Coalfield at Lady Windsor Colliery, as gateroad pack support at advancing longwall faces. The use of narrow coal rib pillars allowed 6 faces rather than 4 with conventional wide pillars, to be extracted in this restricted area of the Four Feet Seam (Fig. 6). Strata dip was on average 6° but seam depth altered from 550m-700m due to the variable surface topography. The immediate roof strata consisted of 4m of silty mudstone which then became interbedded with sandstones and siltstones. A thin 0.6m layer of weak seatearth was underlain by silty mudstones and siltstones with no pronounced bedding planes or stratification as at Betws or Taff Merthyr Collieries. As the coal seam roof contact was fractured with no discernable parting plane, 0.2m of roof coal was left with 1.60m of seam extracted to improve faceline strata control.

Figure 6 - 40s Districts Layouts
Lady Windsor Colliery

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PACK DESIGN

Initial pack support design for all the faces consisted of wood 0.9m chocks with 2.0m infill of bagged loose coal or ripping material. Half bell arches with a running bar on the roadside wood chock and timber lagging supported the gateroad. This system provided adequate support and gateroad conditions for the 41 District, although severe pack compaction had led to the formation of ribside breaks. As 43 Face advanced alongside 41 Panel with the intervening 5m rib pillar, 43 Tailgate suffered from arch distortion, pillar extrusion and floor heave, disrupting face supplies and requiring continual gateroad maintenance. To improve conditions produced by support overloading, the rib pillar was reduced to 3m, and pack stiffness increased by replacing the conventional pack with Aquapak monolithic pump pack (Fig. 7(a)). The wasteside wood chock was retained to maintain immediate backhole support with the Aquapak replaced in 2.0m x 2.0m x 2.5m brattice bags and the half bell arch running bar supported on false legs. Subsequently, pillar width was reduced further to 2.0m, and new brattice bag design and splayed leg arches introduced to control pillar and pack extrusion.

RESULTS

Fully instrumented investigation sites were installed in the Aquapak sections of 43 and 44 Tailgates L.W.1.1 and L.W.2.1 respectively, with photographic profiles plus permanent support surveys supplementing results from investigation sites.

As both faces were longwall advancing, monitoring of the effect of front abutment was not possible. However, visual observations in the adjacent gateroads showed severe deformation and closure occurring 25m in advance of the faceline. In 43 Tailgate Aquapak section, due to emplacement problems, a 0.3m gap resulted which produced a very high rate of initial roof lowering over the gateroad and pack. The only

Site LW.1.1

Site LW.2.1

(a) Gateroad Permanent Support Systems

100m 43 Face Advance

(b) Support Load Development

(c) Proposed Characteristic Deformation Behaviour

Figure 7 - Gateroad Support, Loads and Deformation, Lady Windsor

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resistance to such movement was the wasterside chock taking an initial load of 1 MPa (Fig. 7b). On lowering of the roof onto the Aquapak, the load increased very rapidly to 2.5 MPa with negligible pack compaction. Yield of the fractured roof strata produced a further but gradual increase which stabilised at 3.6 MPa with the wasterside chock at 2.2 MPa. Localised floor failure was not indicated by these results. Levelling surveys conducted to the strata bolts in the gateroad gave a total of 31% vertical closure, of which only 6% was floor heave. Gateroad deformation, monitored using photographic profiling, showed plastic behaviour of the floor strata with no buckling or fractures which may have been aggravated by water pumped from pump packing operations.

With the Aquapak installed tight to the roof at Site L.W.2.1 in 44 Tailgate, rapid initial roof lowering was prevented. The pack gradually compacted and hardened, taking increased load from 0.5 MPa on emplacement to a constant maximum of 1.9 MPa. The wasterside chock, being weaker, took a much reduced load to a constant maximum of 0.8 MPa. As with 43 Tailgate, no reduction in support load occurred indicating that no localised floor failure had occurred. Pillar load, measured by a hydraulic pressure cell installed in a slot in the yielding pillar, also recorded a gradual controlled increase up to 0.8 MPa.

Levelling surveys of the strata bolts showed that of the total 20% gateroad vertical closure, only 2% was floor heave, considerably less than at L.W.1.1 due to the lower support loads and drier floor conditions. Analysis of the 3-D probe results indicated that the majority of floor strata movement occurred within the top 1m of floor strata with 0.4m on the pillars side and 0.3m on the packside. However, strata movements were detected under the supports to over 8.0m depth with high stresses transferred to the lower strata. Gateroad deformation from photographic profiles and proposed characteristic deformation behaviour (Fig. 7c) show that floor strata movement appeared to be a plastic response to evenly distributed loads under the Aquapak and pillar.

CONCLUSIONS

Pack stability governs roof bed tilt and competence, ribside extrusion and depth of yield zone, and ribside roof fracture. Hence, pack design is the key element in ensuring high performance from gate roadway support systems. However, soft floors induce pack instability and special provisions are necessary in such situations.

A feature common to the three pack designs reported is that of the provision of lateral restraint. At Betwe, the RBJ girder raft fulfilled a dual role acting as a tie-bar system and load distributor for the elements of the composite pack. The restraint provided by the pump pack and the splayed leg arches at Taff Merthyr and Lady Windsor respectively, reduced extrusion of the roadides with improved pack and pillar stability.

Where compatible with floor strengths, or even after a level of floor penetration has occurred, monolithic packs sustain uniformly higher loads than alternative forms of pack material such as stone and wood.

The quality of pack installation must be consistently high to generate design stiffness for early and long term loadbearing. In this context, roof bed competence may only be maintained by pack emplacement filling the entire pack hole cavity.

In order to minimise floor heave in gate-roads, pack design must be compatible with ribside supports and floor strata to ensure uniform load distribution across roadway width. However, it is recognised that in very soft floors and with current levels of technology, rapid repair procedures are essential for the removal of the inevitable floor heave.

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APPENDIX 1 - Monolithic Packing Systems (After Clark & Newson, 1985)

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IND: Indefinite