Structural Analysis and Design of Seals for Coal Mine Safety


July 2014

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Structural Analysis and Design of Seals for Coal Mine Safety


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Abstract

This report describes structural analysis and design methods applied to coal mine seals. Three characteristics of coal mine seal design make it challenging and unique, i.e., the pressure-time curves specified for coal mine seal design, the “elasticity of design” requirement, and the seal foundation. An important protective structure design principle to apply in coal mine seal design is that a design should not fail suddenly in a catastrophic brittle mode, but should fail gradually in a ductile mode.

A three-step design procedure for coal mine seals is presented that follows design codes and design criteria developed by the military for design of protective structures. The procedure involves (1) design inputs where the design pressure-time curve, material properties, and seal geometry are specified, (2) foundation design where shear forces around the seal perimeter and the required seal anchorage are determined, and (3) seal structure design where the seal thickness and internal seal reinforcement are determined. Design charts for these seal types are developed for the 830-kPa (120-psi) pressure-time curves with instantaneous rise time. The reinforced concrete designs in the chart are able to withstand a worst-case detonation wave; however, the designs are no longer elastic and permanent deformation occurs.

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Preface

This study was conducted for the National Institute for Occupational Safety and Health, Pittsburg Research Laboratory, under Interagency Agreement IAA-08-GSL-01. The Technical Monitor was Dr. Karl Zipf.

The work was performed by the Structural Mechanics Branch (GS-M) of the Geosciences and Structures Division (GS), U.S. Army Engineer Research and Development Center, Geotechnical and Structures Laboratory (ERDC-GSL). At the time of publication, Dr. Gordon W. McMahon was Chief, CEERD-GS-M; Bartley P. Durst was Chief, CEERD-GS; and Pamela G. Kinnebrew, CEERD-GV-T, was the Technical Director for Survivability and Protective Structures. The Deputy Director of ERDC-GSL was Dr. William P. Grogan, and the Director was Dr. David W. Pittman.

COL Jeffrey R. Eckstein was the Commander of ERDC, and Dr. Jeffery P. Holland was the Director.
## Unit Conversion Factors

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<td>newtons</td>
</tr>
</tbody>
</table>
1 Introduction

1.1 Background

Seals are permanent barriers constructed in underground coal mines to separate abandoned mine workings from active mine workings. Their main purpose, from a safety standpoint, is to prevent methane gas that seeps from coal seams in the abandoned area from intruding into active working areas. They are also designed to contain blast pressures from a possible accidental detonation of the contained gas, or from a coal dust explosion within the sealed area.

Prior to 2006, seals were built to withstand an explosion pressure of 20 psi. This criterion was based on tests conducted at the National Institute for Occupational Safety and Health (NIOSH) Lake Lynn Laboratory’s (LLL) underground test facility, in which candidate seal designs were subjected to a methane gas explosion that produced a peak pressure of at least 20 psi on the seals. If a candidate seal design survived the test, it was approved for use in underground coal mines. Only a limited engineering analysis was used to develop the seal design that was selected.

In 2006, two gas explosion disasters within sealed areas of coal mines in the United States resulted in the deaths of 17 coal miners. After careful consideration of these disasters and gas explosion science, coal mine safety regulations were changed in 2008, and seals are now required to resist an explosion pressure of 120 psi. The new regulations require that licensed professional engineers design seals to withstand a prescribed explosion pressure-time loading curve and that mine management certify construction of the seals according to that design.

1.2 Objectives

Researchers with NIOSH in the Office of Mine Safety and Health Research (OMSHR) developed a research program aimed at eliminating disasters from gas explosions within the sealed areas of coal mines through improved engineering of the entire sealing process. Specific research efforts aim to (1) develop a fundamental scientific understanding of gas accumulation within sealed areas and the explosion pressures developed by gas and dust explosions in coal mines, (2) develop engineering procedures for sealing
abandoned areas of coal mines including structural design of seals and ventilation planning before and after sealing, and (3) develop management systems to control methane accumulation within sealed areas including monitoring procedures and self- and artificial-inertization methods.

Researchers at the U.S. Army Corps of Engineers (USACE) Engineer Research and Development Center (ERDC) are recognized experts in the engineering design of structures that can resist explosion effects and protect military personnel. Because of their recognized expertise, OMSHR researchers developed an interagency research agreement with USACE for guidance in coal mine seal design. The USACE research objective was to apply protective structure design technology used by the defense establishment to the design of seals in the U.S. coal industry and to transfer that technology to the U.S. mining industry.

### 1.3 Scope of work

To meet this objective, ERDC researchers completed four broad tasks.

1. **Back analysis of old seal designs:** Extensive full-scale explosion tests on the old 20-psi seal designs were performed by NIOSH at the Lake Lynn Laboratory over the past 20 years. In this task, the old test results were reexamined to extract useful engineering data for use in the design of new seals that would meet the new 120-psi design standard. Seals made from concrete blocks, cement foam, and polyurethane foam and aggregate were reexamined.

2. **Analysis of 120-psi seals and their components:** After the new seal regulations were promulgated, new seal designs that met the new standards were approved by the Mine Safety and Health Administration (MSHA). In this task, analysis procedures were developed to analyze seal structures including reinforced concrete walls and plain concrete plugs. Methods were also developed to analyze seal foundations, including hitches and rock-bolt anchors.

3. **Application of protective structure design principles to coal mine seal design:** ERDC researchers have been instrumental in developing protective structure design principles and writing several engineering manuals for the design of structures to resist explosion blast effects. The protective structure design principles were applied to the design of coal mine seals. Two detailed seal designs are presented, i.e., reinforced concrete walls and plain concrete plugs.
4. Development of the Wall Analysis Code for Mine Seals (WAC-MS): ERDC researchers previously developed a single-degree-of-freedom analysis program called the Wall Analysis Code (WAC) to perform dynamic structural analyses of reinforced concrete walls subjected to blast loads. This task modified WAC to tailor it for the analysis of coal mine seals. A shear resistance function was developed to perform dynamic structural analyses of plug-like mine seals. Other features were added to enhance the program’s utility for analysis of coal mine seals, and features that did not apply were removed.

1.4 Broad outline of this report

This 10-chapter report documents most of the work conducted in the project to address the scope-of-work outlined above.

1. Chapter 1 introduces coal mine seals and describes the objectives, scope-of-work, and broad outline for the report.

2. Chapter 2 provides background information on coal mine seals. It describes how coal mine seal regulations evolved over the past several decades, and it summarizes the major requirements of the new seal regulations put in place in 2008. The chapter presents a summary of old seal designs that met the old 20-psi criterion along with a summary of MSHA-approved seal designs to meet the new criteria.

3. Chapter 3 describes protective structure design principles and discusses three basic analysis methods used to analyze structures exposed to a dynamic explosion load, i.e., equivalent static method, single-degree-of-freedom analysis, and numerical methods.

4. Chapter 4 examines explosion test data from several old 20-psi seal designs and uses that data to develop a shear resistance function for use in WAC. Specific shear resistance functions are developed for a cement foam plug seal and a polyurethane foam and aggregate plug seal. The shear resistance function also applies to plain concrete plugs.

5. Chapter 5 presents two techniques to anchor a seal structure to the surrounding rock mass, i.e., a hitch and rock-bolt anchors. A hitch is a shallow excavation in the floor rock and coal ribs for anchoring the seal structure. Rock-bolt anchors are embedded into the roof, floor, or ribs and the seal structure itself. Methods to analyze the anchorage capacity of hitches and rock-bolt anchors are presented.

6. Chapter 6 presents methods to analyze two basic kinds of seals, i.e., flexural walls and plugs. The failure modes of both types of seals are described. Detailed examples of analysis calculations for different seal types are also presented.
7. Chapter 7 describes a general procedure for coal mine seal design using protective structure design principles. Detailed examples are provided for the structural design of a reinforced concrete seal and a plain concrete plug, along with their foundations. The designs are for the 120-psi pressure-time curve with instantaneous rise time and with the 0.25-sec rise time. Design charts for reinforced concrete seals and concrete plug seals for both 120-psi design pressure-time curves are also presented that give the required seal dimensions as a function of the seal width and the seal height.

8. Chapter 8 analyzes one reinforced concrete seal design when it is exposed to the explosion pressure from a methane-air detonation wave. The seal design behaves linear elastically when subject to the 120-psi pressure-time curve with instantaneous rise time. When subjected to a normally reflected methane-air detonation wave, the seal structure survives but with permanent deformation.

9. Chapter 9 presents methods to protect seals from the effects of blast waves using blast-wave attenuators. Several simple concepts to construct blast-wave attenuators in mines are described. Preliminary test results demonstrating the effectiveness of blast-wave attenuators are summarized.

10. Chapter 10 summarizes key points and results.

1.5 Important concepts

Numerous techniques and important engineering facts are presented in this study; however, there are three simple overarching concepts that designers of coal mine seals must consider based on the experience of the protective structure design community. First, seal design involves both the design of the seal structure itself and the foundation needed to support or restrain the seal structure. A seal design will fail to perform as intended if it breaks free from the surrounding foundation rock or coal and moves as a unit due to a weak, inadequate foundation design. Second, if a seal is loaded beyond its design capacity, it should fail gradually in a ductile failure mode and not catastrophically through a brittle failure mode. Catastrophic failure through buckling or shear failure must be avoided. Third, seal designs should use materials with known, well-understood, and controllable material properties. For this reason, reinforced concrete is the material of choice for most protective structures in military applications. The rock properties of most seal foundations are unknown and difficult to quantify. For this reason, design engineers should use rock-bolt anchors for the seal foundation rather than relying on unknown frictional properties of the surrounding rock mass.
2 Coal Mine Seals and Seal Regulations

2.1 Evolution of 20-psi coal mine seals and regulations in the United States

At the time of the Sago Mine disaster on 2 January 2006 (Gates et al. 2007), federal mine safety and health regulations required seals to withstand a 20-psi explosion pressure. The earliest seal regulations in the United States trace back to a 1921 regulation governing coal mines located on US government-owned lands that required seals to withstand a pressure of 50 psig. It was “based on the general opinion of men experienced in mine-explosion investigations” (Rice et al. 1931).

The Federal Coal Mine Health and Safety Act of 1969 required mined-out areas to be ventilated or sealed with “explosion-proof bulkheads” that were constructed with “solid, substantial and incombustible materials.” In the publication *Explosion-Proof Bulkheads: Present Practices*, Mitchell (1971) reviewed coal mine explosions and recommended what became the 20-psi criterion for coal mine seals. The general premise behind Mitchell’s recommendation was that coal mine explosions originated in the active areas of coal mines that were ventilated and had only limited quantities of methane or coal dust. An explosion within the sealed area was not considered, because it was commonly believed that sealed areas were inert and either contained methane-rich or oxygen-poor atmospheres. Mitchell reviewed explosion data from hundreds of test explosions conducted in the Bruceton Experimental Mine from 1914 through the 1960s and noted that more than 200 ft from the origin of the test explosion, the pressure seldom exceeded 20 psi. Since most sealed areas are far from the active mining areas, a seal was considered “explosion proof” if it was designed to withstand a static load of 20 psi.

Mitchell developed what became known as the Mitchell-Barrett seal to meet the 20-psi design criterion. This seal consists of two rows of 8-in.-by-8-in.-by-16-in. solid concrete blocks held together with mortar on all sides. It was the most commonly deployed seal in coal mines throughout the 1970s and 1980s. Subsequent tests demonstrated that, in actuality, this 16-in.-thick solid concrete block seal could withstand test explosions of up to about 100 psi.
Prior to 1992, the Code of Federal Regulations (CFR) lacked a definitive design specification for explosion-proof mine seals. Stephan (1990) reviewed Mitchell’s work and also concluded that the explosion pressure on seals generally does not exceed 20 psi. As a result of the Stephan report, the explosion pressure performance criterion for seals became 20 psi in the 1992 change to Code of Federal Regulations Rule 30 CFR 75.335(a)(2). Sometimes this rule change is called the “Alternative Seals Rule,” since it facilitated the development of alternatives to the conventional Mitchell-Barrett seal of solid concrete blocks. Examples of the alternatives include cement foam plug seals, polyurethane foam and aggregate plug seals, Omega Block seals, and wood crib block seals (Zipf et al. 2009). Stephan (1990) recognized that abandoned areas can contain explosive methane-air mixtures when the sealed area atmosphere crosses through the flammable range on its way to becoming methane-rich and inert. Stephan (1990) warned that “a seal constructed to withstand an explosion pressure wave of 20 psi may not be sufficient in these cases.”

To determine whether an alternative seal design met the 20-psi requirement of Rule 30 CFR 75.335(a)(2), the Mine Safety and Health Administration (MSHA) relied on full-scale explosion tests conducted by the National Institute for Occupational Safety and Health (NIOSH) in their Lake Lynn Experimental Mine (LLEM) in Pennsylvania. Working under the direction of MSHA and the seal manufacturers, NIOSH researchers constructed actual full-scale alternative seals at the LLEM and subjected them to a side-on (i.e., quasi-static) pressure of 20 psi that was generated by a test methane explosion. The candidate seal passed the test if it survived the explosion pressure without any visible damage such as cracking or displacement. Air leakage across the seal was then measured to determine if it also met leakage requirements.

At the time of the Sago disaster (2 January 2006), the mining industry had deployed thousands of seals throughout US coal mines that met the 20-psi design standard. On 19 July 2006, MSHA issued a Program Information Bulletin (PIB) No. P06-16, titled “Use of Alternative Seal Methods and Materials Pursuant to 30 CFR 75.335(a)(2)” that required new alternative seals be designed and built to reliably withstand an overpressure of at least 50 pounds per square inch (psi)” (McKinney 2006). Although PIB No. P06-16 raised the seal design standard from 20 to 50 psi, no guidance was given on how new seals should be designed and constructed to meet the new requirement.
The Mine Improvement and New Emergency Response Act of 2006 (the MINER Act), signed into law on 15 June 2006, required mine operators to implement a range of new mandatory mine safety standards. Section 10 of the MINER Act required the Secretary of Labor to “finalize mandatory health and safety standards relating to the sealing of abandoned areas in underground coal mines. Such health and safety standards shall provide for an increase in the 20-psi standard currently set forth in Section 75.335(a)(2) of Title 30, Code of Federal Regulations.”

2.2 NIOSH study of seals and sealed areas of coal mines

Methane gas slowly seeps out of coal seams over a period of time. In working areas of a mine, the normal ventilation prevents any significant buildup of gas. To control methane in mined-out areas of coal mines, mining regulations prior to 2006 (30 CFR 75.334) required mining companies to either ventilate or seal those areas. However, sometime after sealing, the atmosphere within the sealed area may cross through the flammable range when the methane concentration ranges from 5% to 16% and the oxygen concentration exceeds 12%. If the sealed area contains methane and air leakage occurs around seals, a flammable methane-air mixture may form immediately behind the seals. A flammable methane-air mixture might exist in all or part of a sealed area sometime after sealing. If an ignition source is available during this time, the mixture may ignite, and an explosion may develop. The seals must be designed to withstand the explosion pressure generated by such an event and to prevent deadly gaseous explosion products from spreading throughout the mine. Several recent explosions within sealed areas of coal mines resulted in failure of coal mine seals and significant loss of life — most notably, the Sago disaster.

To assist MSHA with its mandate under the MINER Act pertaining to seals, NIOSH conducted a scientific study of the worst-case explosion pressures that could develop within sealed areas of coal mines (Zipf et al. 2007). Instead of assuming that sealed areas were inert, as Mitchell (1971) did, this study recognized that sealed areas could develop the critical methane/air mixture ratio during the transition from an inert low ratio after sealing to an inert high ratio as the methane level increased. The mixture could also return to a critical ratio if the seals leak air into the sealed area at a later time.

NIOSH engineers examined seal design criteria and practices used in the United States, Europe, and Australia and classified seals into their various
applications. Next, they considered various kinds of explosive atmospheres that could accumulate within sealed areas, provided an in-depth phenomenological review of explosion processes, and used thermodynamic calculations and simple gas explosion models to estimate worst-case explosion pressures that could impact seals. The review of methane explosion research cited several studies that recorded methane explosion pressures at or above the detonation pressure, which for a stoichiometric mixture of methane-air is 256 psi.

An explosion in a tunnel behind a mine seal produces an expanding shock wave. The dynamic load on a seal comes from that portion of the shock wave that directly impacts the seal along with the impact of reflections of the shock wave off the walls, floor, and crown of the tunnel. The dynamic load is characterized by a peak pressure and an impulse related to the duration of the load. For simplicity in engineering solutions, these loads can be represented by a set of pressure-time curves.

Based on the worst-case explosion pressures that might have to be resisted, three design pressure-time curves were developed for the dynamic structural analysis of new seals. The three curves were keyed to the conditions under which those seals might be used, i.e., (1) unmonitored seals where there is a possibility of a methane-air detonation, (2) unmonitored seals with little likelihood of a detonation, and (3) monitored seals where the amount of potentially explosive methane-air is strictly limited and controlled. The design blast loading criteria for the three conditions are as follows.

**Condition 1:** For an unmonitored seal with an explosion run-up length of more than 50 m (165 ft), the possibility of detonation or high-pressure nonreactive shock waves and their reflections exists. The recommended design pressure-time curve rises to 4.4 MPa (640 psi) in 0.001 sec and then falls to 800 kPa (120 psig) constant volume (CV) explosion overpressure after another 0.1 sec, where it remains constant for another 4 sec.

**Condition 2:** For an unmonitored seal with an explosion run-up length of less than 50 m (165 ft), the possibility of detonation or high-pressure nonreactive shock waves and their reflections is less likely and a less severe design pressure-time curve applies. The recommended design pressure-time curve rises to 800 kPa (120 psig) in 0.25 sec and then holds at the CV explosion overpressure for 4 sec.
Condition 3: For monitored seals, engineers can use a 345 kPa (50 psig) design pressure-time curve with a rise time of 0.1 sec if monitoring can ensure that (1) the maximum length of explosive mix behind a seal does not exceed 5 m (16 ft) and (2) the volume of explosive mixture behind the seal does not exceed 40% of the total sealed volume. Use of this 345-kPa (50-psig) design pressure-time curve requires continuous monitoring and active management of the sealed area atmosphere.

The NIOSH study also provided simplified seal design charts to demonstrate that reasonably sized seals could be designed to meet the three conditions. However, the seal designs presented did not consider the quality of the surrounding rock mass, the seal anchorage, or any reinforcing steel within the seal. The importance of these factors in a seal’s performance emphasizes the need for more thorough engineering guidelines to design coal mine seals.

2.3 Final rule on sealing of abandoned areas

On 22 May 2007, MSHA issued an “Emergency Temporary Standard (ETS) on Sealing of Abandoned Areas,” which specified new strength requirements of (1) 50-psi (345-kPa) overpressure for sealed areas that are monitored and maintained inert; (2) 120-psi (800-kPa) overpressure if the sealed area atmosphere is not monitored and maintained inert; and (3) greater than 120-psi (800-kPa) overpressure if certain conditions exist within the sealed area that may promote the development of higher explosion pressures (ETS 2007). The ETS became the “Final Rule on Sealing of Abandoned Areas,” which was issued 18 April 2008 and became fully in force by 20 October 2008 (Final Rule 2008).

The major structural engineering requirements for seals in the Final Rule did not change significantly from those in the ETS; however, many requirements were modified or clarified. As stated in the Final Rule summary, “the final rule includes requirements for seal strength, design, construction, maintenance and repair of seals, and monitoring and control of atmospheres behind seals in order to reduce the risk of seal failure and the risk of explosions in abandoned areas of underground coal mines.” It also contains provisions for training of mine personnel conducting work on seals and sealed areas and recordkeeping requirements for archiving data important to seals and sealed areas. This discussion focuses on those requirements in the Final Rule pertinent to the engineering design and analysis of seals.
The seal strength requirements in the form of design pressure-time curves are an important aspect of the Final Rule. Based on the 50- and 120-psi pressure-time curves from the NIOSH study, the Final Rule specifies the following four design pressure-time curves for different applications in underground coal mines.

1. When the sealed area is monitored and maintained inert, the seal design must withstand an instantaneously applied 50-psi overpressure that lasts for 4 sec and is then released instantaneously (30 CFR 75.335(a)(1)(i)). Figure 1 shows this design pressure-time curve for “mainline seals” of sealed areas that are monitored and maintained inert.

![Figure 1. 50-psi design pressure-time curve for sealed areas that are monitored and maintained inert (“mainline seals”).](image)

2. For longwall mining applications where the longwall gob is monitored and maintained inert, the seal design must withstand a 50-psi overpressure that rises in 0.1 sec and then remains at 50 psi (30 CFR 75.335(a)(1)(ii)). Figure 2 shows this design pressure-time curve, which is used for “gob isolation seals” to separate the active longwall panel from the previously mined longwall panel that is monitored and maintained inert.

3. When the sealed area is not monitored and not maintained inert, the seal design must withstand an instantaneously applied 120-psi overpressure that lasts for 4 sec and is then released instantaneously (30 CFR 75.335(a)(2)(i)). Figure 3 shows the design pressure-time curve for “mainline seals” of sealed areas that are not monitored and not maintained inert.
4. For longwall mining applications where the longwall gob is not monitored and not maintained inert, the seal design must withstand a 120-psi overpressure that rises in 0.25 sec and then remains at 120 psi (30 CFR 75.335(a)(2)(ii)). Figure 4 shows this design pressure-time curve, which is used for “gob isolation seals” to separate the active longwall panel from the previously mined longwall panel that is not monitored and not maintained inert.
In addition to the four well-defined design pressure-time curves described above, the Final Rule also has a provision for a pressure-time curve greater than 120 psi (30 CFR 75.335(a)(3)) if the sealed area is not monitored and not maintained inert and if three special conditions are exist. First, the atmosphere in the sealed area must be likely to contain a homogeneous mixture between 4.5% and 17% methane with oxygen greater than 17% (30 CFR 75.335(a)(3)(i)). Second, the mine geometry and layout must be conducive to developing pressure piling should an ignition and explosion occur in the sealed area (30 CFR 75.335(a)(3)(ii)). Third, “other conditions” are encountered that might trigger a detonation in the sealed area (30 CFR 75.335(a)(3)(iii)). The details of this “overpressures greater than 120 psi” pressure-time curve are not defined.

The four pressure-time curves are the main factors controlling the structural design of a coal mine seal under the Final Rule. However, other factors that can have a substantial effect on the seal design are mentioned explicitly in the Final Rule itself or in the associated “Compliance Guide Questions and Answers” (MSHA 2008a) and “Guidelines for the Seal Design Application” (MSHA 2008b). In the Final Rule, “an engineering design application shall address … engineering design and analysis, elasticity of design, material properties, construction specifications, quality control, design references and other information related to seal construction” (30 CFR 75.335(b)(1)).
The “elasticity of design” requirement in the Final Rule imposes the greatest restrictions on seal design. The Guidelines (MSHA 2008b) provide additional guidance on the subject and state that “the results of this analysis demonstrate that the seal design is fully elastic, which allows the seal to withstand repeated overpressure applications.” “Elasticity of design” implies that the seal design must remain within the elastic portion of its load-deformation relationship and that it cannot deform plastically, crack, or undergo any permanent displacements from any single overpressure application.

The “Compliance Guide Questions and Answers” (MSHA 2008a) states that “seals must be able to withstand repeated, independent overpressures from both directions.” This bi-directionality of applied overpressure requirement implies that the seal design must be symmetric, i.e., the seal design must look the same from either the sealed area side or the active mine side since the incoming explosion pressure could impact the seal from either direction.

The Guidelines (MSHA 2008b) provide clarifications of the “construction specifications” requirement in the Final Rule. Each seal design application must “provide information pertaining to the site location and site preparation” and “provide information pertaining to the anchoring of the seal.” For the seal site, construction specifications are “the minimum strata strength requirements for the seal design and the means by which this strength would be determined.” Also relevant to the construction specifications are the requirements to “discuss loose material removal” and “methods for removal of soft floor/fireclay to solid rock.”

Regarding the anchorage of the seal, the Guidelines (MSHA 2008b) request information in the seal design application about the material friction and roughness requirements, if friction is the main method for seal anchorage, or the bar diameter, bar spacing, grout method, and pull test requirements, if steel bars are the main method for seal anchorage, or the depth, width and fill material specifications, if a hitch is the main method for seal anchorage. The development of construction specifications requires the design of a seal foundation that can resist the design pressure-time curve.
2.4 Historic seal designs

From 1992 through 2006, after the alternative seals rule change and before the Sago disaster, NIOSH researchers conducted numerous full-scale tests of various alternative seal constructs to determine whether or not they met the old 20-psi design criterion. Publications by Greninger et al. (1989 and 1991), Weiss et al. (1993, 1996, 1999), Weiss et al. (2002), Sapko et al. (2003), and Sapko, Weiss, and Harteis (2005) describe the various test seals, methods, and results from this 20-psi seal testing program. The applied loading on the seals was measured in all tests, and some included measurements of the seal response as a displacement-time curve. Some of the structural data sets provide a basis for the calibration and verification of numerical models of seal behavior at the 20-psi level, which may, in turn, allow more reliable analyses of seal designs to meet the new 50- and 120-psi explosion pressure design criteria.

The NIOSH “Compendium of Structural Testing Data for 20-psi Coal Mine Seals” (Zipf et al. 2009) organizes and presents the applied loading or P-t curves and, when available, the measured displacement-time (D-t) curves for 44 different seal structures tested prior to 2006 when the former 20-psi design criterion applied to mine seals. Also included in this data set are the applied loading P-t curves and response D-t curves for eight different ventilation stoppings constructed with solid or hollow-core concrete blocks. The structural test results against the stoppings are included as supplemental information pertinent to the design of seals that incorporate concrete blocks in some capacity. The test program was organized into the following six broad categories of seal structures based on the main seal construction material used and the construction method.

**Category 1** includes seals made of concrete or concrete-like materials, such as shotcrete or gunite, with internal steel reinforcement and anchorage to surrounding rock via additional steel reinforcement bars. Seals in this category are the Insteel 3-D Seal (Precision Mine Repair, Inc., Ridgway, Illinois) and the Meshblock Seal (Tecrete Industries Pty. Ltd., New South Wales, Australia, and R. G. Johnson Co., Inc., Washington, Pennsylvania).

**Category 2** includes the so-called pumpable seals constructed with different thicknesses of pumpable cementitious material, depending on the material's compressive strength. Category 2 seals do not contain internal steel reinforcement and are not hitched to the surrounding rock except
through friction between the seal material and the rock. Manufacturers of seals in this category include Minova (Georgetown, Kentucky), HeiTech Corp. (Cedar Bluff, Virginia, and Morgantown, West Virginia), and R. G. Johnson Co., Inc.

**Category 3** seals are “articulated” structures made of discrete concrete blocks, either solid or hollow-core, that may or may not be hitched to the surrounding rock. Seals in this category are the standard solid concrete block seal, which requires hitching, and the solid concrete block seal with Packsetter Bags supplied by Strata Mine Services (Strata Products Worldwide, LLC, Marietta, Georgia), which does not require hitching to withstand 20 psi. Also included in this category are ventilation stoppings designed to withstand a 2-psi overpressure that are made of solid or hollow-core concrete blocks and do not require hitching.

**Category 4** seals are made from a polymer mixed with dry, crushed limestone aggregate, ranging in size from 0.25 to 1 in., placed between two, dry-stacked, hollow-core or solid-concrete-block form walls. This seal does not require hitching. The only example is the MICON 550 seal (MICON, Glassport, Pennsylvania).

**Category 5** seals are made from stacked wood crib blocks nailed together with 4-in.-long nails. These seals are used where high convergence is expected and require hitching into the surrounding rock. This category also includes wood crib block seals that are glued together. The use of Packsetter Bags supplied by Strata Mine Services eliminated the requirement for hitching.

**Category 6** seals are made from lightweight Omega blocks (Burrell Mining Products International, Inc., New Kensington, Pennsylvania) that are cemented together with an MSHA-approved bonding agent called BlocBond, product No. 1225-51, a fiber-reinforced surface bonding cement manufactured by Quikrete Co., Atlanta, Georgia. Lightweight block seals constructed to a thickness of 24 or 32 in. required hitching, whereas lightweight block seals more than 40 in. thick did not require hitching. Seals constructed from lightweight blocks are no longer permitted.

This summary of NIOSH seal tests contains data from a total of 52 different structures including 44 seals and 8 stoppings. Many of the structures were subjected to multiple loadings, and many of the structures were tested to
failure. In some cases, the applied explosion pressure severely damaged the structure or collapsed it completely in the first test. In most cases (36 of the 52), the structures were subjected to repeated explosion loadings. Twenty-eight of the 52 tested structures were loaded to failure. Most of the structures that are considered in this analysis have measured response data in the form of a displacement-time curve developed from a linear variable displacement transducer (LVDT) record.

Of the 52 different tested structures, 41 were evaluated in explosion tests conducted at NIOSH’s Lake Lynn Experimental Mine (LLEM). In addition to the explosion tests, 11 different structures were tested in one of the hydrostatic chambers located in the LLEM, i.e., 9 in the small hydrostatic test chamber and 2 in the large chamber. Of the 15 tests conducted in the small chamber, 5 used water pressure as the loading medium. The other 10 tests used a confined methane-air or similar gaseous mixture explosion within the hydrostatic chamber to develop the test pressure. Both of the tests in the large hydrostatic test chamber used a confined gas explosion to develop the test pressure. One test was conducted in a hydrostatic test chamber in the Safety Research Coal Mine (SRCM) at the NIOSH Pittsburgh Research Laboratory, using water pressure to develop the applied loading.

Of these six categories for the old 20-psi seals, only data from the first four categories have relevance to present coal mine seals. Seals made from concrete (Category 1), cement foam (Category 2), solid concrete blocks (Category 3), or polyurethane foam mixed with aggregate (Category 4) are in use today in some form. Seals made from wood crib blocks nailed or glued together (Category 5) and lightweight cement foam blocks (Category 6) are not used today.

2.5 Summary of approved seal designs

On its website, MSHA has published approved seal designs for 50-psi instantaneous rise time mainline seals, 50-psi slow rise time gob isolation seals, 120-psi instantaneous rise time mainline seals, and four 120-psi slow rise time gob isolation seals (MSHA 2013). At present, no seals have been submitted for approval to meet the greater than 120-psi seal design requirement.
Table 1 summarizes essential characteristics of certain approved seal designs to meet the 120-psi pressure-time curve with an instantaneous rise time using different construction methods and materials.

<table>
<thead>
<tr>
<th>Construction Material</th>
<th>Thickness</th>
<th>Reinforcing Bar</th>
<th>Anchorage Requirements</th>
<th>Convergence Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>3,000-psi concrete</td>
<td>28 in.</td>
<td>#9 bar and others</td>
<td>#9 bar 18-in. deep</td>
<td>Not known</td>
</tr>
<tr>
<td>3,000-psi concrete</td>
<td>8 ft 11 in.</td>
<td>None</td>
<td>Friction only</td>
<td>Not known</td>
</tr>
<tr>
<td>3,000-psi concrete</td>
<td>9 ft 10 in.</td>
<td>None</td>
<td>Friction only</td>
<td>0.192 in. (0.2%)</td>
</tr>
<tr>
<td>400-psi cement foam</td>
<td>15 ft</td>
<td>None</td>
<td>Friction only</td>
<td>19.2 in. (20%)</td>
</tr>
<tr>
<td>Polyurethane foam and aggregate core with concrete block walls</td>
<td>51-in. core 66.5- in. total</td>
<td>None</td>
<td>Friction only</td>
<td>2.5 in.</td>
</tr>
</tbody>
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3 Protective Structure Design and Analysis Methods

This chapter introduces protective structure design concepts that were pioneered by military engineers for the design of structures to resist explosion effects. The principles of protective structure design can be applied to the design of coal mine seals. This chapter discusses three analysis methods that were applied to analyze coal mine seals.

The simplest analysis method is the equivalent static method, in which the given dynamic design problem is transformed into an equivalent static design problem through the use of a dynamic load factor (DLF). This factor converts the dynamic load into an equivalent static load for subsequent analysis and design. The strength of the materials used in the structure are scaled by a dynamic increase factor (DIF) that accounts for the increase in strength that most materials exhibit when subjected to a dynamic load.

The most widely applied method for dynamic structural analysis is numerical solution of single-degree-of-freedom (SDOF) systems. These methods transform the structure into an equivalent mass with an equivalent stiffness and numerically integrate the equation of motion to calculate the displacement response of the structure. The Wall Analysis Code (WAC) (Slawson 1995) is a well-known and widely accepted example of an SDOF analyzer.

Finite element methods were also applied to coal mine seal analysis. When used to conduct a fully dynamic analysis of a structure subjected to a dynamic load, these methods compute the internal dynamic stresses in the structure directly for subsequent design consideration.

This chapter also summarizes the various input parameters that are used in the subsequent analyses. Researchers used the most commonly accepted or a range of commonly accepted material properties for steel, concrete, rock, and other materials used for seal construction.
3.1 Protective structure design

3.1.1 Protective structure design principles

During the last half of the 20th century, the military and military-related groups pioneered detailed engineering procedures for the design of protective structures to shield people and valuable equipment from the effects of explosions. The philosophy and methodology developed for protective structure design also applies to the design of coal mine seals. This section presents the basic concepts of protective structure design and discusses various protective structure design methods in the context of coal mine seal design. Numerous design examples are provided to fully illustrate protective structure design principles.

One overriding concept in protective structure design is the notion of ductile failure as opposed to catastrophic failure. If a structure is subjected to a load beyond its design load, it should not fail catastrophically and disintegrate into fragments with no load-bearing capacity. Rather, the structure should fail in a ductile mode and maintain some of its load-bearing capacity. Beyond the elastic limit of the structure, cracking and plastic deformation should occur gradually and all the while maintaining some load-bearing capacity.

Protective structure design (PSD) involves the conception and planning of buildings and facilities to increase the probability of survival of people or equipment from a threat such as an explosion. Military examples of protective structures include a bunker to protect personnel from a specific explosion threat and assure their survival or a structure to contain an explosion within a part of an explosives storage facility and prevent propagation of the explosion to other parts of the facility. The basic procedure in PSD is threefold.

1. Define the blast load parameters that the required structure must resist.
2. Calculate the dynamic response of various elements in the protective structure such as reinforced concrete or structural steel.
3. Specify the construction details and procedures necessary to develop the required performance of the structural elements to resist the applied blast load.
3.1.2 Design manuals

Prior to the mid 20th century, the design of facilities to resist explosions was empirical; that is, it was based on studies of past catastrophic events. Beginning in the 1960s, military engineers developed quantitative procedures for PSD that are described in several design manuals. Krauthammer (2008) describes manuals developed by the US Department of Defense (DOD), the American Society of Civil Engineers (ASCE), the American Concrete Institute (ACI), and the American Institute of Steel Construction (AISC). The tri-service manual, “Structures to Resist the Effects of Accidental Explosions” (TM 5-1300; Department of the Army, the Navy, and the Air Force 1990), was the most widely used manual in the military and civilian sectors for design of explosion-resistant structures. This manual is available to the public. A related manual, “Fundamentals of Protective Design for Conventional Weapons” (TM 5-855-1; Department of the Army et al. 1998), is not available to the public. TM 5-1300 has been superseded by a new manual also called, “Structures to Resist the Effects of Accidental Explosions” or the Unified Facilities Criteria (UFC 3-340-02; Department of Defense 2008). This manual is also available to the public. These military documents present design procedures for structures that may be subjected to explosions and similar dynamic loads. The design of coal mine seals is a similar problem, i.e., a seal is subject to dynamic loading from a methane-air explosion. The designer must calculate the dynamic response of the seal structure and its foundation, and finally, the designer must specify construction details for each design.

In addition to military design documents, several documents from the civilian sector require consideration for seal design. The American Concrete Institute (ACI) is the primary organization in the US for developing engineering standards for the design and construction of structural concrete used in buildings and other civilian structures. The manual, “Building Code Requirements for Structural Concrete” (ACI 318-08; ACI 2008), specifies all facets of design and construction with this basic building material in all its potential applications. This code describes concrete materials, specifies durability requirements, states quality control procedures, provides construction basics, and states reinforcement standards. The code also describes analysis and design procedures for the main components of concrete structures (columns and beams) and the major forces they must resist, such as flexure, axial loads, shear, and torsion. However, the analysis methods in this design manual are primarily for static loads.
Similar to the ACI, the America Institute of Steel Construction (AISC) is the primary organization in the United States for developing analysis, design, and construction standards for steel used in buildings. The manuals, “Load and Resistance Factor Design – Structural Members, Specifications and Codes” (AISC 1994) and “Steel Construction Manual” (AISC 2005), are the main documents for analysis and design of steel structures. Again, the methods presented in this design code are primarily intended for static loads.

The American Society of Civil Engineers (ASCE) also published design guidelines for various structures subject to dynamic loads. The documents “Design of Blast-Resistant Buildings in Petrochemical Facilities” (ASCE 1997) and “Design of Structures to Resist Nuclear Weapons Effects” (ASCE 1985) offer design guidelines and techniques to resist dynamic loads from explosions that can apply to coal mine seal design.

The study reported here applies the protective structure design methods described in the Unified Facilities Criteria (UFC) 3-340-02 (Department of Defense 2008), which is organized into six chapters.

- Chapter 1 discusses the components of an explosion protection system and the tolerance of humans to explosion forces.
- Chapter 2 describes blast loads from various types of explosions, primary and secondary fragments from explosions, and various kinds of shock loads.
- Chapter 3 presents the principles of dynamic analysis of structures, including resistance functions, dynamically equivalent systems, and dynamic responses.
- Chapter 4 describes reinforced concrete design including the dynamic strength of materials, the design of slabs, beams, and columns, and design for flexure, diagonal shear, and direct shear.
- Chapter 5 discusses structural steel design including the properties of steel, the design of beams and columns, and design to resist failure modes such as tensile failure, shear failure, and buckling.
- Chapter 6 examines special materials for explosive facility design such as masonry, precast concrete, and earth-covered structures.

### 3.1.3 Explosion protection systems

The three basic components to an explosion protection system that interact are the donor system, the acceptor system, and the protection
system itself (UFC 3-340-02). The **donor system** is the size, type, and location of potential explosion sources. The **acceptor system** is the people, equipment, buildings, or nearby explosive stores that need protection. The **protection system** is any type of construction or building, including the acceptor structure itself, and its location that shields the acceptor system from explosion effects to some pre-established level of protection.

For coal mine seal design, the donor system is the methane-air explosion and its underground confinement. The coal mine seal is the protection system that must resist the explosion forces. The Final Rule (2008) specifies explicitly the design pressure-time curve to apply depending on certain conditions. The effects of fragments on the structure, such as the impact of rocks and debris that may be accelerated by the explosion, are not considered.

The acceptor system is also well-defined. It is the people working in the active portion of the coal mine that are to be protected by the seals. The survivability of humans subjected to explosion overpressure is also well understood. For humans protected from blunt force trauma, the threshold for lung damage due to an explosion overpressure is about 10 psi, and 50% lethality results from an explosion overpressure of more than 40 psi (UFC 3-340-02). However, an explosion overpressure of 5 psi is fatal to most exposed humans, primarily due to blunt force trauma-related injuries. While not stated explicitly in the regulations, it is implied that no injury to personnel is acceptable. People on the active mine side of a seal must not be harmed in any way by an explosion that occurs behind the seal. Therefore, the design must ensure that an explosion overpressure at the criterion level cannot breach the seal.

Finally, the protection system is somewhat limited in coal mine seal design. In other applications, the designer can choose the level of protection required depending on the value of the protected contents of the structure and the degree of failure in the protective structure that is tolerable. As discussed in the prior section, the new coal mine seal regulations require that seal designs have “elasticity of design,” meaning that the seal design must remain in the elastic regime when subjected to the design pressure-time curve. No cracking or plastic deformation of the seal is allowed. Therefore, as protective structures, coal mine seals will necessarily provide a very high level of protection.
3.1.4 Explosion pressure design ranges

Conventional protective structure design considers three blast pressure design ranges according to the pressure intensity, i.e., high pressure, low pressure, and very low pressure (UFC 3-340-02). The dynamic response of the protective structure will depend on the magnitude of the applied explosion pressure, the duration of the pressure, and the time required by the structure to reach its maximum deflection, which, in turn, depends on its mass, stiffness, and strength characteristics. For the high-pressure design range, the explosion pressure is much greater than 100 psi, and the pressure duration time, $t_o$, is short compared to the structure response time, $t_m$, which is long. The ratio of the structure’s response time to the pressure duration time, $t_m/t_o$, is greater than 3. For the low-pressure design range, the applied explosion pressure is less than 100 psi, the pressure duration time, $t_o$, is intermediate, and the structure response time, $t_m$, is also intermediate. The ratio $t_m/t_o$ ranges from 0.1 to 3. For the very low pressure design range, the explosion pressure is less than 10 psi, and the pressure duration, $t_o$, is long compared to the structure response time, $t_m$, which is short. The ratio $t_m/t_o$ is less than 0.1.

For structures in the high-pressure design range, the impulse, which is the area under the applied pressure-time curve, controls the structural design rather than the peak pressure. Structures in the low-pressure design range will need to consider both the peak pressure and the impulse in their design. Structures in the very low pressure design range usually only need to consider the peak pressure in their design.

Coal mine seal design does not fit neatly into any one of these UFC pressure design range categories. The peak pressure and pressure duration time, $t_o$, for the pressure-time curves specified in the Final Rule (2008) are 120 psi and 4 sec (4,000 msec), respectively. For a wide variety of coal mine seal designs, including reinforced concrete walls, concrete plugs, and other construction materials, the structure response time, $t_m$, ranges from 10 to 100 msec, and therefore the ratio $(t_m/t_o)$ is most likely much less than 0.1. While the peak pressure is high, coal mine seal design is very similar to protective structure design in the very low pressure design range where peak pressure controls the design. Dynamic effects on the structure are important; however, variation of the applied pressure over time is less important.
3.1.5 Explosion loading categories

In protective structure design, two main blast loading categories are recognized, i.e., unconfined and confined explosions (UFC 3-340-02). Unconfined explosions include “free air bursts” where the explosion is far from any solid surface, “air bursts” where the explosion is within two to three lengths of a structure but too high to produce significant shock wave reflections from the ground surface, and “surface bursts” where the explosion is adjacent to a solid surface so that the incident shock wave loading is amplified by secondary waves reflected from the surface.

For unconfined explosions, the blast loads on the structure result from the impact of the shock waves emanating from the blast. Any high-pressure, gaseous-explosion products exert negligible loading on the structure. Confined explosions occur within some type of full or partial confinement. These include “fully vented explosions” where the gaseous products from the explosion are quickly and totally vented to the atmosphere, “partially confined explosions” where the gaseous explosion products can vent to the atmosphere over a finite time period, and “fully confined explosions” where the gaseous explosion products vent to the atmosphere slowly or not at all. For confined explosions, the blast loads are the initial shock waves from the blast plus the very long-duration pressure from the gaseous explosion products.

For coal mine seal design, the design pressure-time curves specified in the Final Rule (2008) reflect those of a “fully confined explosion” or possibly a “partially confined explosion.” If an explosion developed within the sealed area of a coal mine, the duration of the gas pressure from the explosion products would depend on many factors, such as the volume of the flammable methane-air cloud relative to the sealed volume, the gas pressure containment provided by the rock mass surrounding the sealed volume, and the cooling effectiveness of the rock mass. This duration of the high-gas pressure is expected to be hundreds of milliseconds to seconds in length.

In protective structure design, there are two design ranges for the explosion loads, i.e., the close-in and far design ranges (UFC 3-340-02). The close-in design range arises when high explosives detonate close to a structure and produce a highly non-uniform loading with an extremely high-pressure concentration that can lead to a “punching” failure of a structural component such as a wall. In contrast, the far design range arises when the
explosion is farther away and produces a more uniform distribution of the pressure load over the structure. For most protective design problems, it is assumed that the explosive source is a concentrated mass, such as a stack of ammunition. However in a methane-air detonation, the explosive source is a gaseous mixture that occupies a significant volume rather than a discrete location. Therefore, in coal mine seal design, the explosion load is assumed to be uniform, and the far design range applies even though a portion of the gaseous mixture may detonate adjacent to the seal.

3.1.6 Levels of protection and damage criteria for structures

Reinforced concrete is the preferred material for protective structures because it is low in cost and its structural behavior is well understood. Failure of reinforced concrete elements occurs in stages that depend heavily on the type of shear reinforcement. Figure 5 shows the stages of failure for a reinforced concrete element responding in flexure to a uniform blast load. The deflection is the displacement of the structure at mid-span in the direction of the applied blast load. The support rotation in degrees is calculated from the displacement as

\[ \theta = a \tan \left( \frac{d}{L/2} \right) \]  

(1)

where

\[ d = \text{mid-span displacement} \]
\[ L = \text{span}. \]

Figure 5. Typical resistance-deflection curve for flexural response of concrete elements (after UFC 3-340-02, Final Rule 2008).
When a flexural reinforced concrete element is loaded, linear elastic deformation occurs up to the elastic deflection limit ($X_E$). At the elastic deflection limit, the reinforcing steel begins to yield, and resistance is constant up to 2 deg of support rotation ($\theta = 2^\circ$). At 2 deg of support rotation, the compression concrete will begin to crush, and the wall will fail unless the element is reinforced with shear stirrups. With shear reinforcement, the wall can continue to respond in a flexural mode by transferring the compressive forces in the concrete to the steel. Walls that contain shear reinforcement can continue to provide resistance up to 6 deg of rotation ($\theta = 6^\circ$). The resistance from 2 deg to about 4 deg of support rotation decreases due to the lack of resistance contributed by the crushed concrete. From about 4 deg to 6 deg of support rotation, the steel strain-hardens and provides additional resistance. At 6 deg of support rotation, the element fails. If lacing reinforcement is used, the element will survive until the tensile reinforcement fails at 12 deg of support rotation ($\theta = 12^\circ$).

The UFC 3-340-02 subdivides the protection afforded by a facility into four categories. Protection Category 1 facilities are designed to protect personnel from blast pressures and structural motion; this category applies to coal mine seals. The other protection categories apply to equipment protection and explosive storage. The basic philosophy for protective design is to allow for some level of inelastic response in order for the structure to absorb energy in the plastic deformation region. The level of protection required (high, medium, or low) establishes how much inelastic deformation is allowable.

Figure 6 shows the support rotation criteria used to judge structural failure. For a Protection Category 1 structure, such as coal mine seal with no shear reinforcement, the maximum allowable support rotation for design is 1 deg.

Response limits for acceptable level of damage and level of protection are also found in “Single Degree of Freedom Structural Response Limits for Antiterrorism Design” (PDC-TR 06-08; U.S. Army Corps of Engineers Protective Design Center 2006). Both PDC-TR 06-08 and UFC 3-340-02 are in agreement; however, PDC-TR 06-08 characterizes the level of damage using the more widely recognized “level of protection” terminology.
The Final Rule requires that a seal design remain elastic when subjected to the design pressure-time curve. Therefore, the maximum deflection for a coal mine seal must be less than the elastic deflection limit ($X_{e}$), which is generally much less than a support rotation of 1 deg. By the terminology in PDC-TR 06-08, coal mine seals provide a “high level of protection” primarily because of the “elasticity of design” requirement in the Final Rule.

The Facility and Component Explosive Damage Assessment Program (FACEDAP; Oswald 1993) uses both a ductility ratio and a deflection-to-span-length ratio as criteria to quantitatively define damage and assess the level of protection provided by a structure. These criteria can be used for various types of structural elements and materials including reinforced concrete. Using the FACEDAP criteria for a high level of protection, the allowable response limit in terms of support rotation is 0.23 deg for a one-way slab and 0.48 deg for a two-way slab. For a 10-ft span, these support rotations translate into a center deflection of 0.24 in. and 0.5 in., respectively.

Figure 7(a) shows a 10-ft-high, one-way slab prior to subjecting it to a blast loading. In (b), the blast loading has induced a support rotation of 0.30 deg, and no cracking has occurred. When the support rotation reached 0.90 deg in (c), cracking is barely visible. The “elasticity of design” requirement for coal mine seals in the Final Rule implies that support rotation angles must be much less than 1 deg and that no cracks will be visible.
3.2 Equivalent static analysis method of analysis

In the conventional design of structures subjected to static loads, the applied static load on a structure is compared to the allowable static load. The allowable load considers the strength of the materials used in construction of the structure and strength reduction factors that are based on the kind of stress applied to the material and the expected failure mode. The basic approach to static design, as seen in conventional building codes established by the ACI and the AISC, is to (1) assume a trial design for the structure, (2) calculate the applied loads and stresses within the trial design, (3) calculate the allowable loads and stresses within the trial design, and (4) compare the calculated applied loads and stresses to the calculated allowable loads and stresses for the trial design. The design of structures subject to dynamic loads proceeds along the same general path, except that the applied loads and stresses are now time-varying in nature, such as those loads due to an explosion.

The simplest method for the design of a structure subject to a dynamic load is the equivalent static analysis method in which the applied dynamic load is transformed into an equivalent static load using a dynamic load factor (DLF). The DLF depends on the shape of the applied dynamic load-versus-
time curve and accounts for inertial effects in the design. The allowable loads and stresses are also modified to consider dynamic effects using a **dynamic increase factor** (DIF). The procedure, in effect, reduces the dynamic design problem to an equivalent static design problem through the use of the DLF on the applied loads and stresses and the DIF on the allowable loads and stresses.

### 3.2.1 The Dynamic Load Factor (DLF)

The equation of motion for a single-degree-of-freedom (SDOF) structure is

\[ F - Kx - cv = Ma \]  \hspace{1cm} (2)

where

- \( F \) = applied load as a function of time
- \( K \) = stiffness of structure
- \( c \) = damping coefficient
- \( M \) = mass of structure
- \( x, v, a \) = displacement, velocity, and acceleration of structure.

A solution of the equation of motion for an SDOF system is described in UFC 3-340-02 (2008), Krauthammer (2008), Slawson (1995), and other textbooks on structural dynamics such as Biggs (1964). The solution gives the response of the structure to the applied dynamic load in terms of the maximum deflection, \( X_m \), and the time, \( t_m \), to reach this maximum deflection. For linear elastic systems, analytic solutions to the equation of motion have been developed for several simplified dynamic loadings (or pressure-time curves) such as a triangular load, a rectangular load, a step load with a finite rise time, a triangular load with a finite rise time, and a sinusoidal pulse load.

The maximum dynamic deflection is usually normalized by the maximum static deflection, \( X_s \), which is the displacement produced in the structure when the peak load is applied statically. The normalized variable, \( X_m/X_s \), is the **dynamic load factor** (DLF). In linear elastic systems, deflections, forces, and stresses are all linearly proportional; therefore, the DLF can be applied to any of these quantities to determine the ratio of dynamic-to-static effects.
The time to maximum deflection, $t_m$, (also called the time of maximum response or simply the response time) is normalized by either the load duration, $T$, or the rise time, $T_r$, of the applied dynamic load.

Design charts expressing the non-dimensional DLF and the non-dimensional, normalized response time, $t_m/T$ or $t_m/T_r$, are available for various simplified dynamic loading or pressure-time curves. Two charts of particular interest to coal mine seal design are included from UFC 3-340-02. For an elastic, SDOF system subjected to a rectangular load with zero rise time and finite load duration, $T$, the non-dimensional DLF and the non-dimensional response time versus the non-dimensional time ratio, $T/T_N$, where $T_N$ is the natural period of the structure, are shown in Figure 8. For an elastic, SDOF system subjected to a gradually applied load, the non-dimensional DLF and non-dimensional response time versus the non-dimensional time ratio, $T_r/T_N$, are in Figure 9.

Based on Figures 8 and 9, it is possible to estimate the DLF and the response time for the four possible pressure-time curves specified in the Final Rule. These curves are summarized in Table 2.

As a first approximation, the natural period of a structure, $T_N$, is on the order of several times the transit time of a compression wave across the structure. The compression wave speed for concrete and rock-like materials is typically about 10,000 ft/sec; therefore, the transit time for a compression wave across a 10-ft-thick seal is about 1 msec. It follows then that the natural period of most seal structures is on the order of 5 to 10 msec. From past experience with concrete walls and concrete plugs, the natural period for coal mine seals should range from about 5 to 50 msec.

The total load duration, $T$, for the two design pressure-time curves with an instantaneous rise time is 4,000 msec. Based on the likely range for the natural period, $T_N$, lower and upper bound estimates for the ratio $T/T_N$ are therefore 80 to 800, as shown in Table 3. From Figure 8, the DLF is 2, and the time to maximum response, $t_m$, is 0. The equivalent static pressure for design is twice the peak dynamic pressure, and it will likely always remain so in any practical situation.

The rise times, $T_r$, for the two design pressure-time curves with finite rise times are 100 and 250 msec. Based on the likely range for the natural period, $T_N$, lower and upper bound estimates for the ratio $T_r/T_N$ range from
Figure 8. Maximum response of elastic, SDOF system for a rectangular load as a function of total load duration $T$ over natural period of the system $T_N$ (UFC 3-340-02, 2008).
Figure 9. Maximum response of elastic, SDOF system for a ramp load as a function of rise time, $T_r$, over natural period of the system, $T_N$ (UFC 3-340-02, 2008).
Table 2. Summary of the pressure-time curves specified in the Final Rule.

<table>
<thead>
<tr>
<th>Curve</th>
<th>Reference Figures</th>
<th>Peak Pressure (psi)</th>
<th>Rise Time (ms)</th>
<th>Duration (ms)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 – general purpose seal monitored and maintained inert</td>
<td>2-1</td>
<td>50</td>
<td>0</td>
<td>4000</td>
</tr>
<tr>
<td>2 – general purpose seal not monitored and not maintained inert</td>
<td>2-3</td>
<td>120</td>
<td>0</td>
<td>4000</td>
</tr>
<tr>
<td>3 – longwall panel seal monitored and maintained inert</td>
<td>2-2</td>
<td>50</td>
<td>100</td>
<td>4000</td>
</tr>
<tr>
<td>4 – longwall panel seal not monitored and not maintained inert</td>
<td>2-4</td>
<td>120</td>
<td>250</td>
<td>4000</td>
</tr>
</tbody>
</table>

Table 3. Dynamic load factors for the pressure-time curves with zero rise time as specified in the Final Rule.

<table>
<thead>
<tr>
<th>Curve</th>
<th>Peak Pressure (psi)</th>
<th>Rise Time $T_r$ (msec)</th>
<th>Duration $T$ (msec)</th>
<th>Time Ratio $T/T_N$ Lower Bound</th>
<th>Time Ratio $T/T_N$ Upper Bound</th>
<th>DLF</th>
<th>$t_m/T_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50</td>
<td>0</td>
<td>4000</td>
<td>80</td>
<td>800</td>
<td>2.0</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>120</td>
<td>0</td>
<td>4000</td>
<td>80</td>
<td>800</td>
<td>2.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Table 4. Dynamic load factors for pressure-time curves with non-zero rise times as specified in the Final Rule.

<table>
<thead>
<tr>
<th>Curve</th>
<th>Peak Pressure (psi)</th>
<th>Rise Time $T_r$ (msec)</th>
<th>Duration $T$ (msec)</th>
<th>Time Ratio $T/T_N$ Lower Bound</th>
<th>Time Ratio $T/T_N$ Upper Bound</th>
<th>DLF</th>
<th>$t_m/T_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>50</td>
<td>100</td>
<td>4000</td>
<td>2</td>
<td>20</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>4</td>
<td>120</td>
<td>250</td>
<td>4000</td>
<td>5</td>
<td>50</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

about 2 to 50. From Figure 9, the DLF is approximately 1, and the time to maximum response, $t_m$, is also about 1.0, as also shown in Table 4. Because the dynamic load is applied relatively slowly, the equivalent static pressure for design is equal to the peak dynamic pressure, and it will likely always be equal in any practical situation of mine seal loading conceivable. The time of maximum response and the rise time to peak pressure are also equal, meaning that maximum response occurs when the maximum applied pressure is reached.

### 3.2.2 Dynamic Increase Factor (DIF)

The prior discussion showed that the equivalent static load on a structure is up to twice the magnitude of the applied dynamic load. For the 120-psi
pressure-time curve with an instantaneous rise time, the dynamic load factor is 2, and the equivalent static design pressure is 240 psi.

Construction materials subjected to dynamic loading exhibit higher strength than when subjected to static loading. This increase in material strength is a function of the applied strain rate, is expressed as a ratio between the dynamic strength to the static strength, and is called the dynamic increase factor (DIF). As will be discussed later, the DIF for most practical applications ranges from 1 to about 1.25, meaning that the dynamic strength of the material is up to about 1.25 times the static material strength. The following discussions summarize the important mechanical properties of common materials used in protective structures and their strength behavior under dynamic loading conditions.

3.2.2.1 Concrete

Figure 10a shows the typical stress-strain behavior for concrete as it is loaded to failure. The compressive strength of ordinary, commercially available concrete, $f'_c$, typically ranges from 3,000 to 6,000 psi, and the strain at peak strength is about 0.002 in./in. The secant modulus of elasticity for concrete, $E_c$, increases with the compressive strength of concrete and can be approximated by the formula

$$E_c = 33w^{1.5} \left( f'_c \right)^{0.5}$$

(3)

where $w$ is the unit weight of concrete in lb/ft$^3$ (pcf).

The modulus of elasticity increases slightly as the strain rate increases. Figure 11 shows how the DIF and the dynamic compressive strength of concrete, $d'_c$, increase as a function of the strain rate for a typical concrete. At very high strain rates, the DIF can approach 2; however, this high DIF is seldom used in practice. In terms of the compressive strength of concrete, $f'_c$, Table 5 summarizes the important strength properties and DIF for concrete subjected to different types of stress.
Figure 10. Typical stress-strain curves for concrete (a) and reinforcing steel (b) (UFC 3-340-02, 2008).

(a) STRESS-STRAIN CURVE FOR CONCRETE

(b) STRESS-STRAIN CURVE FOR STEEL
Figure 11. DIF for ultimate compressive strength of concrete (2,500 psi < \( f_c' \) < 5,000 psi) as function of strain rate (UFC 3-340-02, 2008).

Table 5. Dynamic increase factors and concrete strength under different types of stress (UFC 3-340-02, 2008).

<table>
<thead>
<tr>
<th>Type of Stress</th>
<th>Static Strength for Design</th>
<th>Typical Value for Static Strength</th>
<th>DIF ( \left( \frac{f_d}{f_c} \right) )</th>
<th>Typical Value for Dynamic Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>( f_c' )</td>
<td>3,000 psi</td>
<td>1.19</td>
<td>3,570 psi</td>
</tr>
<tr>
<td>Tension</td>
<td>0.1 ( f_c' )</td>
<td>300 psi</td>
<td>1.00</td>
<td>300 psi</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>( 2 \left( f_c' \right)^{0.5} )</td>
<td>110 psi</td>
<td>1.10</td>
<td>121 psi</td>
</tr>
<tr>
<td>Compression</td>
<td>( f_c' )</td>
<td>3,000 psi</td>
<td>1.12</td>
<td>3,360 psi</td>
</tr>
</tbody>
</table>

3.2.2.2 Reinforcing steel and rock bolts

Figure 10b shows the typical stress-strain behavior for reinforcing steel meeting ASTM Standard A615. For Grade 60 reinforcing steel, the static yield strength, \( f_y \), and the static ultimate strength, \( f_u \), are 60,000 and 90,000 psi, respectively. Other common grades used for reinforcing steel are Grade 40 and Grade 75 with static yield strengths, \( f_y \), of 40,000 and 75,000 psi, respectively. Modulus of elasticity is always assumed as
29,000,000 psi, and it is hardly affected by higher applied strain rates due to dynamic loads. The steel used for most common rock bolts used in coal mines behaves similarly to that shown in Figure 10b. Grade 60 is a common grade steel for rock bolts, although some grade 40 is also used. Certain specialty rock bolts may use grade 75 steel or better.

Figure 12 shows how DIF increases with strain rate for reinforcing steel and rock bolts. At extremely high strain rates, the DIF can approach 2, but again, in conservative design, this high DIF is not used in practice. Table 6 summarizes the important strength properties and DIF, in terms of the steel’s static ultimate strength, \( f_y \), for reinforcing steel and rock bolts subjected to different types of stress.

3.2.2.3 Structural steel

The typical stress-strain behavior shown in Figure 10b also applies to structural steel. The most common grade of steel is ASTM standard A36 with a minimum yield strength, \( f_y \), of 36,000 psi and minimum ultimate strength, \( f_u \), of 58,000 psi. Other higher-strength steels are available for protective structures, but they are not considered herein.
Table 6. Dynamic increase factor and strength for reinforcing steel and rock-bolt anchors under different types of stress (UFC 3-340-02, 2008).

<table>
<thead>
<tr>
<th>Type of Stress</th>
<th>Static Strength for Design</th>
<th>Typical Value for Static Strength</th>
<th>DIF ( (f_{dy}/f_y) )</th>
<th>Typical Value for Dynamic Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>( f_y )</td>
<td>60,000 psi</td>
<td>1.17</td>
<td>70,200 psi</td>
</tr>
<tr>
<td>Tension</td>
<td>( f_y )</td>
<td>60,000 psi</td>
<td>1.00</td>
<td>60,000 psi</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>0.6 ( f_y )</td>
<td>36,000 psi</td>
<td>1.10</td>
<td>39,600 psi</td>
</tr>
<tr>
<td>Compression</td>
<td>( f_y )</td>
<td>60,000 psi</td>
<td>1.10</td>
<td>66,000 psi</td>
</tr>
</tbody>
</table>

Figure 13 shows how DIF increases with strain rate for A36 structural steel. At extremely high strain rates, the DIF can approach 2, but again, in conservative design, this high DIF is not used in practice.

Figure 13. Dynamic increase factors for yield stresses of structural steel at various strain rates (UFC 3-340-0008).

Table 7 summarizes the important strength properties and DIF for structural steel subjected to different types of stress, in terms of the steel’s static ultimate strength, \( f_y \).
Table 7. Dynamic increase factor and strength for structural steel under different types of stress (UFC 3-340-02, 2008).

<table>
<thead>
<tr>
<th>Type of Stress</th>
<th>Static Strength for Design</th>
<th>Typical Value for Static Strength</th>
<th>( \text{DIF} \left( \frac{f_{dy}}{f_y} \right) )</th>
<th>Typical Value for Dynamic Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>( f_y )</td>
<td>36,000 psi</td>
<td>1.29</td>
<td>46,440 psi</td>
</tr>
<tr>
<td>Tension or Compression</td>
<td>( f_y )</td>
<td>36,000 psi</td>
<td>1.19</td>
<td>42,840 psi</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>0.55 ( f_y )</td>
<td>19,800 psi</td>
<td>1.19</td>
<td>23,560 psi</td>
</tr>
</tbody>
</table>

3.2.2.4 Concrete blocks

Table 8 summarizes the compressive strength, \( f_m' \), of solid concrete blocks. The modulus of elasticity for concrete masonry units, \( E_m \), is given by

\[
E_m = 1000 f_m' \tag{4}
\]

In terms of the compressive strength of concrete, \( f_c' \), Table 9 summarizes the important strength properties and DIF for concrete blocks subjected to different types of stress.

Table 8. Compressive strength for concrete blocks (UFC 3-340-02, 2008).

<table>
<thead>
<tr>
<th>Type of Unit</th>
<th>Compressive Strength ( \left( f_m' \right) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hollow Units</td>
<td>1,350 psi</td>
</tr>
<tr>
<td>Hollow Units Filled with Grout</td>
<td>1,500 psi</td>
</tr>
<tr>
<td>Solid Units</td>
<td>1,800 psi</td>
</tr>
</tbody>
</table>

Table 9. Dynamic increase factors and concrete block strength under different types of stress (UFC 3-340-02, 2008).

<table>
<thead>
<tr>
<th>Type of Stress</th>
<th>Static Strength</th>
<th>Typical Value for Static Strength</th>
<th>( \text{DIF} \left( \frac{f_{dc}'}{f_c'} \right) )</th>
<th>Typical Value for Dynamic Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure</td>
<td>( f_c' )</td>
<td>1,800 psi</td>
<td>1.19</td>
<td>2,140 psi</td>
</tr>
<tr>
<td>Shear</td>
<td>( f_c' )</td>
<td>1,800 psi</td>
<td>1.10</td>
<td>1,980 psi</td>
</tr>
<tr>
<td>Compression</td>
<td>( f_c' )</td>
<td>1,800 psi</td>
<td>1.12</td>
<td>2,010 psi</td>
</tr>
</tbody>
</table>
3.2.3 Summary

Structures subjected to dynamic loading exhibit higher strength than when they are subjected to ordinary static loading because material strength increases as a function of the applied strain rate. This increase is expressed as a ratio of the dynamic strength to the static strength and is called the dynamic increase factor (DIF). Typically, the DIF is less than 20%, and for concrete and steel, and it depends on the type of stress that is applied, i.e., bending, tension, shear, or compression. UFC 3-340-2 (2008) presents values for concrete and steel based on strain rate and stress type. These values can be used when performing an equivalent static analysis or when using either SDOF or advanced computational methods.

3.3 Numerical solution of SDOF systems with Wall Analysis Code (WAC)

The Wall Analysis Code (WAC; Slawson 1995) is a single-degree-of-freedom (SDOF) model for the analysis of blast-loaded walls as idealized in Figure 14. The first step in a WAC analysis is to calculate the resistance function, \( r \), that describes the load-deformation behavior of a wall given its geometric dimensions, material properties, and support conditions. Next, the wall load-deformation model is transformed to an equivalent SDOF model by the use of SDOF transformation factors. Actual and SDOF-equivalent or transformed loads are calculated based on the weight of explosive and the range to the detonation, or an alternative, user-defined loading may be used. Finally, the equation of motion is solved by numerical integration to determine the response-time history of the equivalent system represented by a critical point on the wall, usually at mid-height and mid-length.

3.3.1 Derivation of WAC

The following derivation from Slawson (1995) starts with the general differential equation for an SDOF system and derives equations suitable for a numerical evaluation.

SDOF methods model the response of a structural element as a spring-mass system. The mass is the mass of the element. The spring stiffness describes the resistance of the responding element to deformation due to the applied loading. The resistance function may be linear, bi-linear (elastic-perfectly plastic), or multi-linear. The general form of the differential equation describing an SDOF system is
\[ m \cdot \ddot{y}(t) + c \cdot \dot{y}(t) + r(y(t)) = f(t) \]  

where

- \( m \) = the mass of the structural element
- \( y(t) \) = the displacement of the mass as a function of time, \( t \)
- \( \dot{y}(t) \) = the velocity of the mass, i.e., the first derivative of displacement with respect to time \( \frac{dy}{dt} \)
- \( \ddot{y}(t) \) = the acceleration of the mass, i.e., the second derivative of displacement with respect to time \( \frac{d^2y}{dt^2} \)
- \( r \) = the structural resistance, which is a function of displacement, \( y \), and the structural stiffness, \( k \)
- \( c \) = the mass damping factor
- \( f(t) \) = the forcing function or structural load

Figure 14. SDOF idealization of a blast-loaded wall. (Slawson 1995).
The effective mass and load for SDOF systems based on the deformed shape of the wall and loading distribution are defined as mass \((K_M)\) and load \((K_L)\) factors and are calculated as

\[
K_M = \frac{\int m(x) \cdot \varphi^2(x) \, dx}{\int m(x) \, dx}
\]  

(6)

where \(\varphi(x)\) is the shape function describing the deformed shape of the structural element as a function of the distance, \(x\), from the first support, normalized so that the maximum deflection equals 1, and \(m(x)\) is structural mass as a function of \(x\).

\[
K_L = \frac{\int p(x) \cdot \varphi(x) \, dx}{\int p(x) \, dx}
\]  

(7)

where \(p(x)\) is the load as a function of distance from the support and is typically uniformly distributed. Using Equations 6 and 7 and defining the load-mass factor \((K_{LM})\) as the ratio of the mass and load factors \((K_M/K_L)\), Equation 5 can be rewritten as

\[
K_{LM} \cdot M_t \cdot \ddot{y}(t) + K_{LM} \cdot c \cdot \dot{y}(t) + R(y(t)) = F(t)
\]  

(8)

where \(M_t\) is the total structural mass, \(R(y(t))\) is the total resistance of the wall, which is a function of the wall deformation history, and \(F(t)\) is the total wall loading as a function of time. By substituting \(M = K_{LM} \cdot M_t\), \(C_d = K_{LM} \cdot cK_{LM} \cdot c\), \(F_l = R(y(t))\), and \(F_E = F(t)\), Equation 8 can be rewritten in the general form of

\[
M \cdot \ddot{y}(t) + C_d \cdot \dot{y}(t) + F_I = F_E
\]  

(9)

To calculate the damping coefficient, the critical damping ratio is determined by Equation 10.

\[
C_{cr} = 2 \cdot \sqrt{k \cdot m}
\]  

(10)

where \(k\) is the wall stiffness and \(m\) is the mass of the wall. The damping coefficient, \(C_d\), is taken as 0.5% of the critical value (if very lightly damped, \(c = 0.005 \cdot C_{cr}\)).
The equation of motion is numerically integrated in time using a central difference scheme. The standard damped central difference integration scheme (Equation 11 for acceleration, velocity, and total displacement) was recast in incremental form as shown in Equation 12 by substituting \( y_{t+\Delta t} \) for \( y_{t+\Delta t} \) and \( y_{t-\Delta t} \) for \( y_{t+\Delta t} \) (where \( \Delta \) indicates an increment). Equation 12 calculates the incremental displacement (\( \Delta y_{t+\Delta t} \)) from time equals \( t \) (the beginning of the current time step) to time equals \( t+\Delta t \) (the end of the current time step).

\[
\dot{y}_t = \frac{1}{\Delta t^2} \cdot (y_{t-\Delta t} - 2 \cdot y_t + y_{t+\Delta t})
\]

\[
\ddot{y}_t = \frac{1}{2 \cdot \Delta t} \cdot (y_{t+\Delta t} - y_{t-\Delta t})
\]  

(11)

\[
y_{t+\Delta t} = C_{\beta} \cdot \left( \frac{F_E - F_i}{M} \cdot \Delta t^2 \cdot y_{t-\Delta t} \cdot \left[ 1 - \frac{C_{\alpha} \cdot \Delta t}{2} \right] + 2 \cdot y_t \right)
\]

where

\[
C_{\alpha} = C_{d} \cdot M
\]

\[
C_{\beta} = \frac{2}{2 + C_{\alpha} \cdot \Delta t}
\]

\[
\Delta y_{t+\Delta t} = C_{\beta} \cdot \left( \frac{F_E - F_i}{M} \cdot \Delta t^2 + \Delta y_t \cdot \left[ 1 - \frac{C_{\alpha} \cdot \Delta t}{2} \right] \right)
\]  

(12)

Since the central difference integration scheme is conditionally stable, the \( \Delta t \) (time step) selected must satisfy stability criteria to obtain a convergent solution. The stability criterion is based on the natural period of the wall (\( \Delta t_{T_n} \)). The undamped natural period of the wall may be approximated by using the stiffness and mass as shown in Equation 13.

\[
T_n = 2 \cdot \pi \cdot \sqrt{\frac{M}{k}}
\]  

(13)

The undamped natural period of the wall is used in calculating the critical time step as shown in Equation 14. This relationship is conservative since
the incremental central difference method is stable for time steps less than $T_d/2\pi$ ($T_d$ is the damped natural period).

$$\Delta t_n = \frac{1}{10} \cdot T_n$$ (14)

In addition to the time step requirements for stability, the time step should also be much less than the duration of the applied load to ensure that the loading history is adequately represented. This is critical for very impulsive (short duration) loads. A $\Delta t$ that provides 100 time steps or more during the duration of an impulsive load is usually adequate. WAC uses a minimum of 200 time steps during the positive phase duration.

This method of solution of the equation of motion has proven very reliable in other SDOF codes and in multi-degree-of-freedom codes (Slawson 1995).

### 3.3.2 Flexural resistance function for WAC

WAC has five basic resistance function options for walls; however, only two are relevant to this study, i.e., (1) the flexural resistance function for reinforced concrete and masonry walls and (2) a user-defined option in which a shear resistance function was developed for this study.

Flexural walls and their related resistance functions are classified as either one-way or two-way and with simple, fixed, or free support conditions. In a one-way wall, the structure can flex in only one direction and is completely unrestrained in the other. For a coal mine seal, a one-way wall could be attached to the coal ribs and be unrestrained at the roof and floor, or the seal could be attached to the roof and floor and unrestrained by the ribs. For practical coal mine seal analysis and design, a one-way flexural analysis may not be the most realistic.

For a two-way wall, the structure can flex in both directions, and for a coal mine seal, that means that it can flex both rib to rib and roof to floor. This situation is generally more realistic for practical coal mine seal analysis and design.

A simple support condition means that the connection between the structure and its foundation, i.e., the seal and the roof, rib, or floor rock, can rotate but cannot resist any bending moments, whereas a fixed support condition resists all bending moment and cannot rotate at all. A free
support condition means that the connection can translate laterally and cannot resist either bending or rotation. In practical coal mine seal design and analysis, a free support condition is not recommended. During seal design and subsequent construction, all four edges of the seal perimeter must be attached to the surrounding foundation rock through direct contact with cohesion and friction, with an excavation (hitch) into the surrounding rock, or with anchors of some kind. Support conditions are usually neither perfectly simple nor perfectly fixed but have some component of each. For most practical coal mine seal applications, the simple support condition with no moment resistance is closest to reality since the foundation rocks can deform and cannot prevent support rotation.

For coal mine seal analysis and design, the following cases could apply.

- **One-way walls:**
  - Case 11 – simple-simple
  - Case 12 – fixed-fixed
  - Case 13 – fixed-simple (propped cantilever)
  - Case 14 – fixed-free (cantilever)

- **Two-way walls with two adjacent sides supported:**
  - Case 21 – fixed-fixed-free-free (two adjacent sides fixed)
  - Case 22 – simple-fixed-free-free (fixed along long dimension, width)
  - Case 23 – fixed-simple-free-free (fixed along short dimension, height)
  - Case 24 – simple-simple-free-free (two adjacent sides simply supported)

- **Two-way walls with three sides supported:**
  - Case 31 – fixed-fixed-fixed-free (three sides fixed, one free)
  - Case 32 – fixed-simple-fixed-free
  - Case 33 – simple-fixed-simple-free
  - Case 34 – simple-simple-simple-free (three sides simply supported, one free)

- **Two-way walls with four sides supported:**
  - Case 41 – fixed-fixed-fixed-fixed (all sides fixed)
  - Case 42 – simple-fixed-simple-fixed (top and base fixed, sides simple)
Case 43 – fixed-simple-fixed-simple (sides fixed, top and base simple)
Case 44 – simple-simple-simple-simple (all sides simple)

For practical coal mine seal analysis and design, one-way walls (Cases 11, 12, 13 and 14) are not likely nor are they desirable because two of the seal edges are free to translate. Similarly, two-way walls with only two or three sides supported are undesirable, again because certain seal edges are free to translate. Only two-way walls with four sides supported should be considered for practical coal mine seals and analyses. The most desirable situation to achieve is Case 41 where all four seal edges are fixed and able to resist moment without rotation; however, achieving truly fixed end conditions is difficult in practice. The most likely situation to occur is that of Case 44 with all edges simply supported.

The SDOF resistance functions for one-way reinforced walls may be determined from Table 10. The SDOF transformation factors, $K_L$ and $K_M$ for one-way reinforced walls, are shown in Table 11. Figure 15 shows a typical resistance function with the different response regimes delineated.

### Table 10. Resistance functions for one-way reinforced elements (Slawson 1995).

<table>
<thead>
<tr>
<th>Case</th>
<th>Description</th>
<th>Elastic Stiffness, psi/in.</th>
<th>Elastic Resistance, psi</th>
<th>Ultimate Resistance, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>simple-simple</td>
<td>$\frac{384 \cdot E \cdot I}{5 \cdot L^4}$</td>
<td>$\frac{8 \cdot M_p}{L^2}$</td>
<td>$\frac{8 \cdot M_p}{L^2}$</td>
</tr>
<tr>
<td>12</td>
<td>fixed-fixed</td>
<td>$\frac{384 \cdot E \cdot I}{L^4}$</td>
<td>$\frac{12 \cdot M_N}{L^2}$</td>
<td>$\frac{8 \cdot (M_N + M_p)}{L^2}$</td>
</tr>
<tr>
<td>13</td>
<td>fixed-simple</td>
<td>$\frac{185 \cdot E \cdot I}{L^4}$</td>
<td>$\frac{8 \cdot M_N}{L^2}$</td>
<td>$\frac{4 \cdot (M_N + 2 \cdot M_p)}{L^2}$</td>
</tr>
<tr>
<td>14</td>
<td>fixed-free</td>
<td>$\frac{8 \cdot E \cdot I}{L^4}$</td>
<td>$\frac{2 \cdot M_N}{L^2}$</td>
<td>$\frac{2 \cdot M_N}{L^2}$</td>
</tr>
</tbody>
</table>

where

$E$ = Young’s modulus for concrete, $(153 \cdot UWT)^{\prime \prime \prime}$, where UWT is the unit weight

$I$ = effective moment of inertia of the wall section per unit width

$M_N$ = ultimate negative moment capacity (compression on inside face) per unit width at the supports

$M_p$ = ultimate positive moment capacity (compression on outside face) per unit width at mid-height
Table 11. Transformation factors for one-way reinforced elements (Slawson 1995).

<table>
<thead>
<tr>
<th>Case</th>
<th>Range of Behavior</th>
<th>Load Factor, $K_L$</th>
<th>Mass Factor, $K_M$</th>
<th>Load-Mass Factor, $K_{LM}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>Elastic Plastic</td>
<td>0.64</td>
<td>0.50</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.50</td>
<td>0.33</td>
<td>0.66</td>
</tr>
<tr>
<td>12</td>
<td>Elastic Elasto-Plastic Plastic</td>
<td>0.53</td>
<td>0.41</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.64</td>
<td>0.50</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.50</td>
<td>0.33</td>
<td>0.66</td>
</tr>
<tr>
<td>13</td>
<td>Elastic Elasto-Plastic Plastic</td>
<td>0.58</td>
<td>0.45</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.64</td>
<td>0.50</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.50</td>
<td>0.33</td>
<td>0.66</td>
</tr>
<tr>
<td>14</td>
<td>Elastic Plastic</td>
<td>0.40</td>
<td>0.26</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.50</td>
<td>0.33</td>
<td>0.66</td>
</tr>
</tbody>
</table>

For two-way walls, the UFC 3-340-02 approach is taken to determine the resistance functions for walls supported on four sides, three sides, or two adjacent sides. This approach includes calculating the ultimate capacity using yield line theory (symmetric yield line patterns only) and calculating
load-deformation points up to ultimate resistance using incrementally elastic solutions.

The yield line method assumes that, after initial cracking of the concrete at points of maximum moment, yielding spreads until the full (or effective) moment capacity is developed along the length of the cracks on which failure takes place. The following rules apply for valid yield line patterns.

1. To act as plastic hinges of a collapse mechanism made up of plane segments, yield lines must be straight lines forming axes of rotation for the movements of the segments.
2. The supports of the wall will act as axes of rotation. A yield line may form along a fixed support.
3. For compatibility of deformations, a yield line must pass through the intersection of the axes of rotation of the adjacent wall segments.

UFC 3-340-02 recommends a reduction in moment capacity for yield lines near the corners of two-way elements. Effective moment capacities are taken as two-thirds of the full moment capacity in regions near corners. The reduced moment capacity is considered over the regions shown in Figure 16. The first step in performing a yield line analysis is to assume a yield line pattern. Valid yield line patterns for the support cases considered are shown in Figure 17. Ultimate resistances for the two-way wall cases are determined by solving the equations shown in Table 12 where the moment capacity locations are defined in Figure 16.

Figure 16. Regions where ultimate moment capacity is reduced in yield line analysis (Slawson 1995).
Definitions of the variables used in Table 12 are given below and in Figure 16.

\[ L = \text{length of the element (in.)} \]
\[ H = \text{height of the element (in.)} \]
\[ x = \text{yield line location in the horizontal direction (in.)} \]
\[ y = \text{yield line location in the vertical direction (in.)} \]
\[ M_{YN} = \text{ultimate negative moment capacity in the vertical direction (in.-lb)/in.)} \]
\[ M_{VP} = \text{ultimate positive moment capacity in the vertical direction (in.-lb)/in.)} \]
\[ M_{HN} = \text{ultimate negative moment capacity in the horizontal direction (in.-lb)/in.)} \]
\[ M_{HP} = \text{ultimate positive moment capacity in the horizontal direction (in.-lb)/in.)} \]
Table 12. Ultimate resistances for two-way support cases (Slawson 1995).

<table>
<thead>
<tr>
<th>Edge Conditions</th>
<th>Yield Line Locations</th>
<th>Limits</th>
<th>Ultimate Resistance (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two adjacent sides supported and two sides free</td>
<td><img src="image1" alt="Diagram" /></td>
<td>$x \leq L$</td>
<td>$\frac{5(M_{HN} + M_{HP})}{x^2}$ or $\frac{6L M_{VN} + (5M_{VP} - M_{VN})x}{H^2(3L - 2x)}$</td>
</tr>
<tr>
<td></td>
<td><img src="image2" alt="Diagram" /></td>
<td>$y \leq H$</td>
<td>$\frac{5(M_{VN} + M_{VP})}{y^2}$ or $\frac{6H M_{HN} + (5M_{HP} - M_{HN})y}{L^2(3H - 2y)}$</td>
</tr>
<tr>
<td>Three sides supported and one side free</td>
<td><img src="image3" alt="Diagram" /></td>
<td>$x \leq L/2$</td>
<td>$\frac{5(M_{HN} + M_{HP})}{x^2}$ or $\frac{2M_{VN}(3L - x) + 10xM_{VP}}{H^2(3L - 4x)}$</td>
</tr>
<tr>
<td></td>
<td><img src="image4" alt="Diagram" /></td>
<td>$y \leq H/2$</td>
<td>$\frac{5(M_{VPN} + M_{VP})}{y^2}$ or $\frac{4(M_{HN} + M_{HP})(6H - y)}{L^2(3H - 2y)}$</td>
</tr>
<tr>
<td>Four sides supported</td>
<td><img src="image5" alt="Diagram" /></td>
<td>$x \leq L/2$</td>
<td>$\frac{5(M_{HN} + M_{HP})}{x^2}$ or $\frac{8(M_{VN} + M_{VP})(3L - x)}{H^2(3L - 4x)}$</td>
</tr>
<tr>
<td></td>
<td><img src="image6" alt="Diagram" /></td>
<td>$y \leq H/2$</td>
<td>$\frac{5(M_{VPN} + M_{VP})}{y^2}$ or $\frac{8(M_{HN} + M_{HP})(3H - y)}{L^2(3H - 4y)}$</td>
</tr>
</tbody>
</table>
The equations from Table 12 are solved simultaneously for the location of the yield line (x or y), and the ultimate resistance is calculated by substitution of the yield line location into either equation.

The initial portion of the resistance function (Figure 15) is composed of an elastic region and one or more elasto-plastic ranges. The elastic resistance \( r_e \) is defined as the resistance at which the first yield occurs. The elasto-plastic regions are determined by subsequent yielding at additional yield line locations. The ultimate resistance \( r_u \) is the resistance when all the yield lines have formed. When \( r_u \) is reached, the wall is fully plastic, and no further increase in resistance is considered (no tensile membrane action). In addition, no explicit consideration of compressive membrane action or post-ultimate degradation of resistance is considered for reinforced walls. The elastic, elasto-plastic, and ultimate resistances and deformations are determined by the use of Figure 18 for two-way walls with four sides supported (the corresponding UFC 3-340-02 figure numbers are in parenthesis above each figure).

The following discussion illustrates the procedure for determining the resistance function for a two-way reinforced concrete wall with fixed supports on all edges.

Figure 19 shows the initial conditions of the two-way wall with all edges fixed.

- Step 1 – Calculate moment capacities (MVP, MVN, MHP, MHN).
- Step 2 – Calculate the yield line location (x) and the ultimate resistance \( r_u \) using Table 12.
- Step 3 – Calculate the elastic deflection \( y_e \) and resistance \( r_e \) using Figure 18 Case 41.

Negative yield lines have formed along the top and bottom edges of the wall resulting in support conditions of sides fixed, top and bottom simply supported (Case 43) as shown in Figure 20.

- Step 4 – Calculate the elasto-plastic deflection \( y_{ep} \) and resistance \( r_{ep} \) using Figure 18 for Case 43.
Figure 18. Moment and deflection coefficients for two-way walls with four sides supported (Slawson 1995).

Cases 42 and 43 [Fig 3-34]
Two Opposite Sides Fixed, Two sides Simply Supported

Case 41 [Fig 3-33]
Four Sides Fixed
Negative yield lines have now formed along all supports, resulting in support conditions of simple all sides (Case 44) as shown in Figure 21.

- Step 5 – Calculate the remaining points on the resistance function (assume only one more point in this example so that only the deformation ($y_p$) at ultimate resistance must be determined) using Figure 18 for Case 44. Note that there may be one or two more points depending on when the positive yield lines form. In each step above, yielding at positive and negative yield line locations are checked to calculate additional resistances and corresponding elasto-plastic deformations to define the resistance function. The resistance function is shown in Figure 22.

In reality, there could possibly be several elasto-plastic load-deflection points depending on when the positive yield lines formed.
Figure 21. Yield lines have formed along all supports (Slawson 1995).

Figure 22. Resistance function (Slawson 1995).

The elastic and elasto-plastic load-mass transformation factors ($K_{LM}$) for two-way walls are given below in Table 13. Plastic transformation factors for two-way walls supported on all sides are taken from Biggs (1964) rather than the curve given in UFC 3-340-02.

Section properties for reinforced concrete and masonry walls are determined from Jones (1989). The moment and inertia used in calculating the stiffness of one-way walls is the average of the gross and cracked moments of inertia. Young’s modulus of concrete is calculated as

\[ E_c = 33 \cdot \gamma_c^{1.5} \cdot \sqrt{f_{c'}'} \]  

(15)

where

\[ \gamma_c = \text{unit weight of concrete (pcf)} \]

\[ f_{c'}' = \text{unconfined compressive strength of concrete (psi)} \]
Table 13. Elastic and elasto-plastic load-mass factors for two-way walls (Slawson 1995).

<table>
<thead>
<tr>
<th>Edge Conditions</th>
<th>Values of L/H</th>
<th>All Supports Fixed</th>
<th>One Support Simple, Other Supports Fixed</th>
<th>Two Supports Simple, Other Supports Fixed</th>
<th>All Supports Simple</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two adjacent sides supported and two sides free</td>
<td>All</td>
<td>0.65</td>
<td>0.66</td>
<td>--</td>
<td>0.66</td>
</tr>
<tr>
<td>Three sides supported and one side free</td>
<td>L/H ≤ 0.5</td>
<td>0.77</td>
<td>0.77</td>
<td>0.79</td>
<td>0.79</td>
</tr>
<tr>
<td>L/H ≤ 0.5</td>
<td>0.65</td>
<td>-0.16(L/2H - 1)</td>
<td>0.66</td>
<td>-0.144(L/2H - 1)</td>
<td>0.65</td>
</tr>
<tr>
<td>L/H ≥ 2</td>
<td>0.65</td>
<td>0.66</td>
<td>0.65</td>
<td>0.66</td>
<td>0.66</td>
</tr>
<tr>
<td>Four sides supported</td>
<td>L/H = 1</td>
<td>0.61</td>
<td>--</td>
<td>0.62</td>
<td>0.63</td>
</tr>
<tr>
<td>L/H = 1</td>
<td>0.61</td>
<td>+0.16(L/H - 1)</td>
<td>--</td>
<td>0.62</td>
<td>0.63</td>
</tr>
<tr>
<td>L/H ≥ 2</td>
<td>0.77</td>
<td>--</td>
<td>0.78</td>
<td>0.79</td>
<td></td>
</tr>
</tbody>
</table>

The standard Hognestad stress-strain relation for concrete shown in Figure 23 is used for calculating section moment capacities.

Figure 23. Stress-strain relation for concrete used in WAC (Slawson 1995).
3.4 Shear resistance function for WAC

The WAC was originally developed for analysis of frontal loads, and the associated resistance functions are frontal in nature. The typical application for the WAC is a front-loaded wall that fails in flexure, where the response is calculated using a flexural resistance function. Figure 24, modified from Figure 14, shows the equivalent SDOF system considered by WAC for a front-loaded wall or seal acting in flexure. Flexural resistance functions for walls are built into WAC, and their use is transparent to the user.

\[ P(t) = \text{frontal load} = p(t) \cdot W \cdot H \]  \hspace{1cm} \text{(16)}

where

\( P(t) = \) design pressure-time curve or measured pressure-time curve  
\( W = \) wall width  
\( H = \) wall height

\[ M = \text{mass of wall or seal} = \gamma \cdot W \cdot H \cdot T \]  \hspace{1cm} \text{(17)}

where

\( \gamma = \) density  
\( T = \) wall thickness

The flexural resistance of the wall or seal, \( R_f \), depends on the stiffness, \( k \), and the frontal area, \( W \cdot H \).

The WAC has the option for a user-defined resistance function. A shear resistance function for analysis of thick walls and plugs using the SDOF
method is developed as follows. Figure 25 shows the equivalent SDOF system for a front-loaded wall or seal where shear resistance around the perimeter resists the applied frontal load.

Figure 25. Equivalent SDOF system for front-loaded wall or seal with shear resistance around the perimeter.

The applied frontal load on the seal is first transformed into an equivalent shear load acting around the perimeter of the seal. The applied frontal load, $P(t)$, in Figure 24 is equal to the shear load, $P_s(t)$, acting around the perimeter of the seal as shown in Figure 26.

Figure 26. Relationship between $p(t)$, $p_s(t)$, and $r(t)$. 
\[ P(t) = P_s(t) \]  

(18)

With Equation 16, the applied frontal pressure times the frontal area must equal the applied shear-stress times its contact area given by perimeter thickness.

\[ P(t) = p(t) \cdot W \cdot H = p_s(t) \cdot T \cdot (2 \cdot W + 2 \cdot H) = P_s(t) \]  

(19)

where

\[ p_s(t) = \text{equivalent applied shear stress acting around the seal perimeter.} \]

The applied shear stress around the perimeter area is thus

\[ p_s(t) = p(t) \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)} \]  

(20)

Equation 20 gives the applied shear stress versus time function that is input to WAC for an SDOF analysis.

The stability of plug seals is based on balancing the load applied to the face of the seal against the shearing resistance at the seal-rock interface. From static analysis, the applied frontal load must equal the shear load acting around the perimeter which, in turn, must equal the shear resistance, \( R_s(t) \), at the seal-rock interface.

\[ P(t) = P_s(t) = R_s(t) \]  

(21)

With Equation 19, the applied frontal pressure times the frontal area must equal the applied shear-stress times its contact area, which, in turn, must equal the shear resistance stress, \( r_s(t) \), times its contact area given by perimeter thickness.

\[ p(t) \cdot W \cdot H = p_s(t) \cdot T \cdot (2 \cdot W + 2 \cdot H) = r_s(t) \cdot T \cdot (2 \cdot W + 2 \cdot H) \]  

(22)

The shear-stress resistance function at the seal-rock interface is therefore
\[ r_s(t) = p(t) \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)} \]  \hspace{1cm} (23)

where

\[
p(t) = \text{pressure acting in the front face of the plug} \\
W = \text{width of the plug} \\
H = \text{height of the plug} \\
T = \text{thickness of the plug}
\]

As will be demonstrated later, this shear-stress resistance function is determined from an experiment that results in shear failure of the plug structure or seal.

The shear strain for a plug is calculated as

\[ \varepsilon_s(t) = \frac{\Delta(t)}{T} \]  \hspace{1cm} (24)

where

\[ \Delta(t) = \text{plug displacement.} \]

Figure 27 shows an example of a shear-stress resistance function for a typical plug-type seal. This function is the shear-stress resistance function that is input to the WAC for an SDOF analysis.

The applied shear stress versus time curve, given by Equation 20, and the shear-stress resistance function, given by Equation 23, act on the seal-rock interface whose area is determined as

\[ \text{Area} = T \cdot (2 \cdot W + 2 \cdot H) \]  \hspace{1cm} (25)

For input to the WAC, an equivalent set of seal dimensions, \(W'\) and \(H'\), is required such that the equivalent frontal area is equal to the seal-rock interface area.

\[ W' \cdot H' = T \cdot (2 \cdot W + 2 \cdot H) \]  \hspace{1cm} (26)

For convenience,

\[ W' = H' \]  \hspace{1cm} (27)
An equivalent set of seal dimensions, $W'$ and $H'$, for input to WAC when conducting an SDOF analysis in shear is thus

$$W' = H' = \sqrt{T \cdot (2 \cdot W + 2 \cdot H)}$$

(28)

The mass of the seal is still computed with equation 17 above. However, when using the equivalent dimensions $W'$ and $H'$, an effective density, $\gamma'$, is used in the WAC.

$$M = \text{mass of wall or seal} = \gamma \cdot W \cdot H \cdot T = \gamma' \cdot W' \cdot H' \cdot T$$

(29)

The effective density for input to the WAC when conducting an SDOF analysis in shear is therefore

$$\gamma' = \gamma \cdot \frac{W \cdot H}{W' \cdot H'}$$

(30)
The discussion in this section provides the mechanical background for how to develop a shear resistance function for an SDOF analysis using the WAC. Chapter 4 presents experimental data and develops shear-stress resistance functions for certain plug-like seals.

3.5 Finite element methods of analysis

Depending on the need, three different finite element methods were applied during the course of these studies, i.e., LS-DYNA, HONDO, and ABAQUS. Each tool has its own unique strengths and capabilities. LS-DYNA and ABAQUS are commercially available, but HONDO is a research tool used in certain government laboratories.

3.5.1 LS-DYNA

LS-DYNA is an advanced general-purpose multi-physics simulation software package developed by the Livermore Software Technology Corporation (Livermore Software Technology Corporation (LSTC) 2012). While the package continues to contain more and more possibilities for the calculation of many complex, real world problems, its origins and core-competency lie in highly nonlinear transient dynamic finite element analysis (FEA) using explicit time integration. LS-DYNA is used by the automobile, aerospace, construction, military, manufacturing, and bioengineering industries. This makes it a good tool for mine seal analysis since it is available both to industry and government. LS-DYNA was used to analyze reinforced concrete seals, plain concrete plug seals, and the foundations for these seals. These analyses provided a detailed understanding of the material behavior and failure modes of the seals and their foundations.

LS-DYNA is a general purpose, transient dynamic, finite element program capable of simulating complex real world problems. The code was developed specifically to perform analyses in the very dynamic realms of impact, blast loadings, and structural responses. It is optimized for shared and distributed memory, UNIX-, Linux-, and Windows-based, platforms. LS-DYNA has almost 100 constitutive models to simulate a range of engineering materials, from steels to composites and soft foams to concrete. The new version, 971, features updated models for shell and solid elements including the new aluminum honeycomb materials. The code has an extensive element library including membrane, thin-shell, thick-shell and solid formulations. Special features in the code include automatic contact
algorithms, airbag inflation, seat belt elements, and deformable to rigid switching. The 971 version also offers an option for use on parallel computing systems. The lower-order finite elements in LS-DYNA are accurate, efficient, simple, and fast. For the under-integrated shell and solid elements, zero-energy modes are controlled by either hourglass viscosity or stiffness.

Seal structures modeled with LS-DYNA consisted of eight-node solid elements. Loads were applied as pressure boundary conditions to the seals, i.e., a pressure-time history was applied to the faces of specific element faces to simulate the loading cases. Boundary conditions were simulated by applying fixity to certain nodes, i.e., constraining motion in certain directions to simulate the restrained edges of a model including “fixed” walls, ceilings, and symmetry planes.

An elastic-plastic material model was used for any steel rebar contained within a seal. This model explicitly defined the yield strength, the elastic (Young’s) modulus, Poisson’s ratio, a post-yield work hardening curve, and a failure level for effective plastic strain. When an element reaches that effective plastic strain limit, it is eroded from the calculation process simulating a “failure” of the steel at that location within the model. A strain-rate enhancement curve taken from TM 5-855-1 (Departments of the Army et al. 1998) was used as typical for all the steel models, conservatively providing a higher stress level as the strain rates increased within the simulations.

All geo-materials used, including concrete and the *in situ* materials, were modeled using the HJC constitutive model (Holmquist et al. 1993) that contains strength, damage, strain rate effects, and a hydrostatic pressure-volume relationship. Volumetric strain, effective plastic strain, and pressure all contribute to the damage calculated within the material. The HJC model was developed for large-strain, high-strain rate, and high-pressure problems, such as the response to blast or impact loading. Included within the formulation are material damage, strain-rate effects, and permanent crushing as a function of the pressure and air void ratio. Damage is accumulated from both equivalent plastic strain and plastic volumetric strain. A pressure-volume relationship is established based on three regions, i.e., an initial linear elastic region up to a crush pressure, a transition region between the crush pressure and the locking pressure, and the third region where the air voids have been completely removed and the material is “locked.”
3.5.2  HONDO

HONDO is a large-deformation, elastic or inelastic, dynamic finite element method computer program used to calculate time-dependent displacement, velocity, acceleration, and stress within two-dimensional or axisymmetric solids. Hondo features several unique material models, and these capabilities were used to analyze the behavior of plug seals and seals constructed with mine waste rock (gob). The original version of HONDO was developed at Sandia National Laboratories (Key et al. 1978), but the version used in these studies is from the Air Force Weapons Laboratory (Merkle and Dass 1985) and has been further modified by researchers at USACE for specific applications. HONDO is not commercially available to the public.

The original version of HONDO incorporated an elastic-plastic model with strain-hardening and strain-rate behaviors, visco-elastic behavior, rubber elasticity, and soil and crushable foam behavior. The Air Force Weapons Laboratory constitutive model (Merkle and Dass 1985) used in these studies is a non-associative, elastic-perfectly-plastic model with one yield surface, as shown in Figure 28. The yield surface is a series of straight lines in the stress space $\sqrt{J_2}$ versus $I_1$, where $J_2$ is the second invariant of the deviatoric stress tensor, and $I_1$ is the sum of the principal stresses. This yield surface has a Drucker-Prager portion where the yield stress increases with friction, followed by a von Mises portion where the yield stress is constant. The model also has a tensile strength cut-off.

Figure 28. Failure surface for aggregate skeleton model (Merkle and Dass 1985).
Table 14 summarizes the important material parameters used in this model, as illustrated in Figure 28. The minimum laboratory testing needed to define these parameters are (1) a hydrostatic (isotropic) compression test to measure the bulk modulus during both loading and unloading, (2) a uniaxial strain test to measure the Poisson’s ratio during loading and unloading, and (3) at least two triaxial compression tests, or at least two pure shear tests at constant height, to determine the yield surface for the model. This yield surface is defined by the Drucker-Prager intercept $Y$ at $I_1 = 0$, also called the cohesion, and the limit value for the deviatoric stress $J_2$. These tests will also determine the slope or frictional properties of the material in the Drucker-Prager portion of the model.

<table>
<thead>
<tr>
<th>Type Response</th>
<th>Parameter</th>
<th>Term</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pressure-Volume Response</td>
<td>$\mu$</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_v$</td>
<td>Volume strain</td>
</tr>
<tr>
<td></td>
<td>$K_i$</td>
<td>Loading bulk modulus at $\varepsilon_v$</td>
</tr>
<tr>
<td></td>
<td>$K_u$</td>
<td>Unload/reload bulk modulus at pressure $P_i$</td>
</tr>
<tr>
<td>Deviatoric Plastic Response</td>
<td>ST</td>
<td>Apparent tensile strength (negative)</td>
</tr>
<tr>
<td></td>
<td>$Y$</td>
<td>Drucker-Prager intercept at $I_1 = 0$ (Cohesion)</td>
</tr>
<tr>
<td></td>
<td>$S$</td>
<td>Slope of Drucker-Prager yield surface in $(J_2')^{1/2}$ vs. $I_1$ space</td>
</tr>
<tr>
<td></td>
<td>VM</td>
<td>Limit value of $(J_2')^{1/2}$ for the von Mises yield surface</td>
</tr>
<tr>
<td>Notes</td>
<td>$\varepsilon_v$</td>
<td>Volumetric strain</td>
</tr>
<tr>
<td></td>
<td>$P$</td>
<td>$(\sigma_1 + \sigma_2 + \sigma_3)/3 = \text{mean stress}$</td>
</tr>
<tr>
<td></td>
<td>$J_2'$</td>
<td>Second invariant of deviator stress tensor</td>
</tr>
<tr>
<td></td>
<td>$I_1$</td>
<td>$\sigma_1 + \sigma_2 + \sigma_3 = \text{trace of stress tensor}$</td>
</tr>
</tbody>
</table>

### 3.5.3 ABAQUS

ABAQUS is a commercially available finite element method computer program for calculating the time-dependent displacement and stress within two-dimensional, three-dimensional, or axisymmetric solids. The program features capabilities for (1) pre-processing or modeling to create the input file and set up the problem for analysis, (2) solving problems using implicit or explicit integration, and (3) post-processing to display results with images or animation. The solver includes comprehensive
element and constitutive model libraries for linear, non-linear, and contact problems both static and dynamic.

### 3.5.4 Material properties used in the constitutive model

The constitutive models used in the finite element programs LS-DYNA and HONDO require material properties data derived from testing to determine the constitutive model’s parameters. For the analyses of coal mine seals, material properties are needed for the concrete, coal, and rock, and the mine waste rock. Specifically, the constitutive models need a failure surface, in terms of the stress difference ($J_2$) versus normal stress ($I_1$), a pressure versus volumetric strain curve that determines the loading and unloading bulk modulus, and other parameters, including the bulk shear modulus and the Poisson’s ratio. Schwer (2001) provides a comprehensive description of this testing and its application to geo-material constitutive models.

For concrete materials, several fits to standard concrete mixes have been shown to produce good results. For these analyses, a fit to a concrete material designated SAM-35 was used. This is a typical 3500-psi concrete that has been well characterized. A complete set of the required material properties tests were performed on this mix (Williams et al. 2006). Table 15 summarizes the important material properties used in the constitutive model of this concrete.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Concrete (SAM-35)</th>
<th>Weak Shale (Rock 1)</th>
<th>Competent Shale (Rock 2)</th>
<th>Strong Coal (Coal 4)</th>
<th>Mine Waste (gob)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\mu$ – Poisson’s ratio</td>
<td>0.11</td>
<td>0.387</td>
<td>0.49</td>
<td>0.42</td>
<td>0.25</td>
</tr>
<tr>
<td>$K_l$ – Bulk modulus (loading)</td>
<td>$1.5 \times 10^6$</td>
<td>$6.43 \times 10^4$</td>
<td>$2.25 \times 10^6$</td>
<td>$1.17 \times 10^6$</td>
<td>800</td>
</tr>
<tr>
<td>$K_u$ – Bulk modulus (unloading)</td>
<td>$12.6 \times 10^6$</td>
<td>$1.1 \times 10^6$</td>
<td>$2.25 \times 10^6$</td>
<td>$1.17 \times 10^6$</td>
<td>2.0 $\times 10^4$</td>
</tr>
<tr>
<td>ST – Tensile strength, psi</td>
<td>-350</td>
<td>-144.0</td>
<td>-204.0</td>
<td>-900.0</td>
<td>-144.0</td>
</tr>
<tr>
<td>Y – Drucker-Prager intercept (cohesion)</td>
<td>1166</td>
<td>215</td>
<td>337</td>
<td>745</td>
<td>215</td>
</tr>
<tr>
<td>S – Slope of Drucker-Prager yield surface</td>
<td>0.2389</td>
<td>0.1194</td>
<td>0.05656</td>
<td>0.3759</td>
<td>0.1194</td>
</tr>
<tr>
<td>VM – limit value of von Mises yield surface</td>
<td>$3.76 \times 10^4$</td>
<td>602</td>
<td>870</td>
<td>$5.0 \times 10^4$</td>
<td>602</td>
</tr>
</tbody>
</table>
The material properties of coal and rock found in coal mines can vary greatly and range from weak, soil-like materials to competent limestone that is much stronger than concrete.

Table 16 (Zipf 2006) presents a range of soil, rock, and coal materials representative of those found in coal mines. Three materials from this table were used as the basis for developing pressure-volume and failure surface curves for the constitutive models in LS-DYNA or HONDO.

Table 16. Summary of coal and rock material properties for modeling and analysis.

<table>
<thead>
<tr>
<th>Material Name</th>
<th>Description</th>
<th>Lab UCS (MPa)</th>
<th>Field UCS (MPa)</th>
<th>Young's Modulus (GPa)</th>
<th>Cohesion (MPa)</th>
<th>Friction Angle (deg)</th>
<th>Dilation Angle (deg)</th>
<th>Tensile Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil 1</td>
<td>paste</td>
<td>0.04</td>
<td>0.02</td>
<td>1</td>
<td>0.007</td>
<td>21</td>
<td>10</td>
<td>0.002</td>
</tr>
<tr>
<td>Soil 2</td>
<td>very soft soil</td>
<td>0.07</td>
<td>0.04</td>
<td>1</td>
<td>0.014</td>
<td>21</td>
<td>10</td>
<td>0.004</td>
</tr>
<tr>
<td>Soil 3</td>
<td>soft soil</td>
<td>0.14</td>
<td>0.08</td>
<td>1</td>
<td>0.028</td>
<td>21</td>
<td>10</td>
<td>0.008</td>
</tr>
<tr>
<td>Soil 4</td>
<td>firm soil</td>
<td>0.29</td>
<td>0.16</td>
<td>1.5</td>
<td>0.055</td>
<td>21</td>
<td>10</td>
<td>0.016</td>
</tr>
<tr>
<td>Soil 5</td>
<td>stiff soil</td>
<td>0.63</td>
<td>0.35</td>
<td>2</td>
<td>0.120</td>
<td>21</td>
<td>10</td>
<td>0.035</td>
</tr>
<tr>
<td>Soil 6</td>
<td>very stiff soil</td>
<td>3.6</td>
<td>2.0</td>
<td>2.5</td>
<td>0.69</td>
<td>21</td>
<td>10</td>
<td>0.20</td>
</tr>
<tr>
<td>Rock 1</td>
<td>claystone, fireclay</td>
<td>6.4</td>
<td>3.6</td>
<td>3</td>
<td>1.2</td>
<td>22</td>
<td>10</td>
<td>0.3</td>
</tr>
<tr>
<td>Rock 2</td>
<td>black shale</td>
<td>11</td>
<td>6</td>
<td>4</td>
<td>2.0</td>
<td>23</td>
<td>10</td>
<td>0.6</td>
</tr>
<tr>
<td>Rock 3</td>
<td>black shale, gray shale</td>
<td>18</td>
<td>10</td>
<td>5</td>
<td>3.3</td>
<td>24</td>
<td>10</td>
<td>1.0</td>
</tr>
<tr>
<td>Rock 4</td>
<td>gray shale</td>
<td>25</td>
<td>14</td>
<td>6</td>
<td>4.5</td>
<td>25</td>
<td>10</td>
<td>1.4</td>
</tr>
<tr>
<td>Rock 5</td>
<td>siltstone, gray shale</td>
<td>34</td>
<td>19</td>
<td>7</td>
<td>6</td>
<td>26</td>
<td>10</td>
<td>1.9</td>
</tr>
<tr>
<td>Rock 6</td>
<td>siltstone</td>
<td>48</td>
<td>27</td>
<td>8</td>
<td>8</td>
<td>28</td>
<td>10</td>
<td>2.7</td>
</tr>
<tr>
<td>Rock 7</td>
<td>siltstone sandstone</td>
<td>63</td>
<td>35</td>
<td>10</td>
<td>10</td>
<td>30</td>
<td>10</td>
<td>3.5</td>
</tr>
<tr>
<td>Rock 8</td>
<td>sandstone, limestone</td>
<td>77</td>
<td>43</td>
<td>12</td>
<td>12</td>
<td>32</td>
<td>10</td>
<td>4.2</td>
</tr>
<tr>
<td>Rock 9</td>
<td>sandstone</td>
<td>95</td>
<td>53</td>
<td>15</td>
<td>14</td>
<td>34</td>
<td>10</td>
<td>5.2</td>
</tr>
<tr>
<td>Rock 10</td>
<td>limestone</td>
<td>139</td>
<td>78</td>
<td>20</td>
<td>20</td>
<td>36</td>
<td>10</td>
<td>7.7</td>
</tr>
<tr>
<td>Coal 1</td>
<td>banded, bright coal</td>
<td>3.6</td>
<td>2.0</td>
<td>2.5</td>
<td>0.6</td>
<td>29</td>
<td>10</td>
<td>0.17</td>
</tr>
<tr>
<td>Coal 2</td>
<td>banded coal</td>
<td>6.3</td>
<td>3.5</td>
<td>2.5</td>
<td>1.0</td>
<td>30</td>
<td>10</td>
<td>0.29</td>
</tr>
<tr>
<td>Coal 3</td>
<td>banded, dull coal</td>
<td>12</td>
<td>6.7</td>
<td>2.5</td>
<td>1.9</td>
<td>31</td>
<td>10</td>
<td>0.60</td>
</tr>
<tr>
<td>Coal 4</td>
<td>dull coal</td>
<td>17</td>
<td>9.7</td>
<td>2.5</td>
<td>2.7</td>
<td>32</td>
<td>10</td>
<td>0.85</td>
</tr>
</tbody>
</table>

Note: UCS = Unconfined Compressive Strength
For modeling the mine waste (gob) material that might be used in seal construction, the material properties for the weak shale listed in Table 15 were modified. Uniaxial compression tests conducted by Pappas and Mark (1993) on rock fragments that simulated mine waste rock at a laboratory scale were the basis for this modification. The material properties for the mine waste rock are also summarized in Table 15.
4 Analysis of Pre-2006 NIOSH and USBM Seal Tests

Using various analysis tools, structural tests of coal mine seals conducted at the NIOSH Lake Lynn Laboratory (LLL) were examined. Adequate data already exist to verify the analysis and design of seals constructed from conventional steel and concrete materials. However, little data exist for the analysis and design of seals constructed with cement foam, polyurethane foam and aggregate, or mine waste rock (gob). The single-degree-of-freedom model Wall Analysis Code (WAC) was applied to test data of the cement foam plug seal and polyurethane foam and aggregate plug seal types. Based on full-scale test data of various seals, resistance functions were developed that describe the response of a seal to a load, either static or dynamic. The resistance functions are based on a shear failure mode around the perimeter of the seal and apply to plug-type seal designs. The resistance functions were verified against the existing test data and then used to design similar structures to resist the new MSHA design loads stated in the Final Rule (2008). These resistance functions were then built into the Wall Analysis Code for Mine Seals (WAC-MS) presented in the Appendix A of this report.

4.1 Analysis of cement foam plug seals

Cement foam plug seals, sometimes called “pumpable” seals, are made from cement foam materials of various compositions. Different formulations produce different compressive strengths for the pumpable cement foam used to construct the seal. The old 20-psi and the new 120-psi cement foam plug seals do not contain any internal steel reinforcement and are not hitched to the surrounding rock. Friction between the seal material and the surrounding rock holds the seal in place.

In this section, the responses of cement foam plug seals tested at LLL are analyzed. The experiments at LLL showed that shear failure around the perimeter of the seal is the failure mode, and they established bounds for maximum shear strength of a cement foam plug seal. Load-versus-deflection data from a static test was used to develop a generic resistance function suitable for use in an SDOF analysis. An analysis of the dynamic tests at LLL using this generic resistance function in the WAC produced
good results when compared to the experimental data. However, for different formulations of cement foam, i.e., ones that have higher or lower strengths, additional test data are required to determine allowable shear strengths and develop a resistance function for use in a dynamic analysis.

The analysis of cement foam plug seals reported here begins by developing a generic shear stress-versus-shear strain resistance function that depends on the shear strength ($\tau_{\text{max}}$) of the cement foam material. Methods to estimate the shear strength of this material are then discussed. The generic shear resistance function is used to calculate the response of a cement foam plug seal for comparison to its measured response to an explosion load. Finally, the generic resistance function is used to calculate the thickness of a cement foam plug seal that meets the new 120-psi design standard for coal mine seals.

### 4.1.1 Shear-stress resistance function for cement foam plug seals

Data from LLL Test C3-44E (Sapko et al. 2005; Zipf et al. 2009) was used to develop an approximate shear-stress resistance function for cement foam plug seals failing along the seal/rock interface. Dimensions of the test seal were 21.2-ft wide by 8.7-ft high by 4.0-ft thick. Test C3-44E was conducted in the small hydrostatic test chamber at LLL. The test seal was loaded by a methane gas explosion in the chamber. The average compressive strength of the cement foam was 350 psi with a standard deviation of 75 psi.

Figure 29 shows the measured pressure-time and displacement-time data for this test. The measured pressure versus displacement data is shown in red in Figure 30. The pressure was applied uniformly to the face of the seal and was quasi-static, since it rose to its peak pressure of 32 psi in about 7 sec. The displacement response was measured with an LVDT gage at the edge of the test seal close to the interface with the surrounding rock. This measured displacement is assumed to characterize the shear displacement of the entire seal structure. The LVDT recorded a maximum displacement of 3.25 in.; however, this was not the true maximum displacement. Additional experimental data showed the maximum displacement to be as much as 5.5 in.

As shown by the test data in Figure 30 (red curve), there is an initial elastic behavior up to about 25 psi at 0.15 in. of displacement, followed by an inelastic region where the seal resistance reaches a peak of 32 psi, and then falls to about 30 psi as the displacement increases to the limit of the
instrumentation. The blue curve in Figure 30 is an approximate fit to the measured pressure-versus-displacement data extended out to 5.5 in. of displacement. The data points for this fit curve are shown in Table 17.

**Figure 29.** Measured pressure-time data and displacement-time data for Test C3-44E.

**Figure 30.** Measured applied pressure-versus-displacement data (in red) for Test C3-44E and fitted approximation (in blue).
The shear-stress resistance at the interface, or perimeter, of the seal is related to the frontal pressure applied to the face of the seal using Equation 23.

\[
  r_s(t) = \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)} = p(t) \cdot \frac{21.2 \cdot 8.7}{3 \cdot (2 \cdot 21.2 + 2 \cdot 8.7)} = 1.03 \cdot p(t)
\]  

where \( r_s(t) \) is the shear-stress resistance at the rock/seal interface and \( p(t) \) is the frontal pressure acting on the seal.

Most materials exhibit a strength increase when subjected to dynamic loads. This strength increase is not reflected in this test data since it was a static loading test. The UFC 3-340-02 (2008) tabulates Dynamic Increase Factors (DIF) for design using common construction materials such as steel and concrete; however, cement foam is not in this list of materials. The DIF for direct shear in conventional concrete is 1.10. This DIF is also assumed to apply for direct shear in cement foam materials. The shear-stress resistant function, adjusted for the strength increase due to dynamic loading, becomes

\[
  r_s(t) = 1.03 \cdot p(t) \cdot DIF = 1.03 \cdot p(t) \cdot 1.10 = 1.13 \cdot p(t)
\]  

The shear strain is calculated using Equation 24, where \( \Delta(t) \) is the measured displacement and \( T \) is the seal thickness. In this experiment, the seal was constructed 4-ft thick; however, only 3 ft of the cement foam plug seal was in contact with the surrounding strata, as shown in Figure 31. The shear strain was therefore calculated with the reduced interface thickness, \( T \), of 3 ft.
Table 18 shows the shear strain computed with Equation 24 and the adjusted shear-stress resistance computed with Equation 32 based on the measured displacement and frontal pressure data given in Table 17. The shear-stress resistance function for this test is shown in Figure 32.

<table>
<thead>
<tr>
<th>Shear Strain, in./in.</th>
<th>Adjusted shear-stress resistance, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00000</td>
<td>0.00</td>
</tr>
<tr>
<td>5.55 \times 10^{-5}</td>
<td>3.81</td>
</tr>
<tr>
<td>0.00527</td>
<td>30.28</td>
</tr>
<tr>
<td>0.01330</td>
<td>34.88</td>
</tr>
<tr>
<td>0.02030</td>
<td>35.93</td>
</tr>
<tr>
<td>0.06810</td>
<td>35.31</td>
</tr>
<tr>
<td>0.15300</td>
<td>27.57</td>
</tr>
</tbody>
</table>

The shear-stress resistance function given in Table 18 is independent of the seal geometry. However, it is specific to cement foam with a uniaxial compressive strength of 350 psi and a maximum shear-stress resistance of 35.93 psi. A generic shear-stress resistance function can be developed by expressing the shear-stress resistance as a function of $\tau_{\text{max}}$. After normalizing the adjusted shear-stress resistance in Table 18 by 35.93 psi, the generic shear-stress resistance as a function of $\tau_{\text{max}}$ is obtained as shown in Table 19.
4.1.2 Estimating maximum shear-stress resistance $\tau_{\text{max}}$

A direct shear test from a material testing laboratory is desirable when determining the maximum shear-stress resistance. In the absence of this data, one could estimate the value by following the procedure explained below.
For common strength concrete, the direct shear capacity is not a linear function of its compressive strength. The most common empirical formula for the shear strength of concrete is

$$\tau_{\text{max}} = n\sqrt{F_c'}$$  \hspace{1cm} (33)

where \(n\) is a coefficient that depends on the type of shear (diagonal or direct). For direct shear in concrete, \(n=8\) with a limit of \(0.35F_c'\) (Krauthammer 2008).

$$\tau_{\text{max}} = 8\sqrt{F_c'} \leq 0.35F_c'$$  \hspace{1cm} (34)

For a low-grade concrete with an \(F_c' = 3,000\) psi, the expected shear strength is about 440 psi. According to the ACI, 400 psi is the minimum shear strength to expect for concrete (ACI 2008).

If we assume that cement foam materials will exhibit a similar type of relationship as concrete, it is possible to estimate a range of values for \(n\). From the LLL experiments on Omega Blocks with compressive strengths of 152 psi, the range for shear-stress capacities was determined to be 20 psi to 75 psi (Walker 2007). The corresponding \(n\) values are 1.6 to 6. Therefore, if no experimental data are available, the maximum direct shear-stress resistance for cement foam materials could be estimated as

$$\tau_{\text{max}} = (1.6 \text{ to } 6)\sqrt{F_c'}$$  \hspace{1cm} (35)

The lower (L) and upper (U) bounds for maximum direct shear-stress resistances are

$$\left(\tau_{\text{max}}\right)_L = 1.6\sqrt{F_c'}$$  \hspace{1cm} (36)

$$\left(\tau_{\text{max}}\right)_U = 6\sqrt{F_c'}$$  \hspace{1cm} (37)

Figure 33 shows a plot of these bounds for \(\tau_{\text{max}}\) as a function of the uniaxial compressive strength of the cement foam.
Experimental data can be used to narrow the gap between these bounds. Based on Equation 20, when a seal is tested to a maximum frontal pressure, \( p(t)_{\text{max}} \), the corresponding maximum shear stress is

\[
\tau_{\text{max}} = p(t)_{\text{max}} \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)}
\]  

(38)

If the seal does not fail during the test, then this value of \( \tau_{\text{max}} \) from Equation 38 is the new experimental lower (E.L.) bound for the shear strength, \( (\tau_{\text{max}})_{\text{E.L.}} \). Therefore the new lower bound is equal to

\[
(\tau_{\text{max}})_{\text{E.L.}} = p(t)_{\text{max}} \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)} \geq 1.6\sqrt{F'_c}
\]  

(39)

If the seal fails during the test, then this value of \( \tau_{\text{max}} \) from Equation 38 is the new experimental upper (E.U.) bound for the shear strength, \( (\tau_{\text{max}})_{\text{E.U.}} \). Therefore, the new upper bound is equal to
\[
(\tau_{\text{max}})_{E.U.} = p(t)_{\text{max}} \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)} \leq 6 \sqrt{F_c}
\]  

(40)

**4.1.3 Validating the shear-stress resistance function for cement foam plug seals**

The generic shear-stress resistance function for a cement foam plug seal presented in Table 19 was validated against test data developed by Weiss and Harteis (2008) at LLL. Figure 34 shows the geometry of this mostly cement foam plug seal, and Table 20 summarizes the dimensions and properties of the seal. The initial 4-ft-thick cement foam plug seal was modified by constructing an 8-in.-thick, solid concrete block wall on the unloaded side of the seal and filling the 5-in.-thick space in between with polyurethane foam. The outside of the block wall was reinforced with a layer of carbon-fiber reinforced polymer. The seal does not match the exact conditions for the generic shear-stress resistance function derived above; however, it is close, and the analysis of this seal illustrates the general utility of the function.

![Figure 34. Cross section of the cement foam seal in LLEM Tests 508 and 509.](image)
Table 20. Dimensions and properties of cement foam plug seal for validating the generic shear-stress resistance function.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width, $W$</td>
<td>18.7 ft (224 in.)</td>
</tr>
<tr>
<td>Height, $H$</td>
<td>7.3 ft (88 in.)</td>
</tr>
<tr>
<td>Total seal thickness</td>
<td>5.08 ft (61 in.)</td>
</tr>
<tr>
<td>Cement foam strength, $F_c'$</td>
<td>143 psi</td>
</tr>
<tr>
<td>Cement foam density</td>
<td>35 pcf</td>
</tr>
<tr>
<td>Concrete block density</td>
<td>140 pcf</td>
</tr>
<tr>
<td>Average seal density</td>
<td>48.77 pcf</td>
</tr>
</tbody>
</table>

4.1.3.1 Analysis of Test 508

Figure 35 shows the frontal pressure versus time data obtained during Test 508. The maximum frontal pressure, $p_{\text{max}}$, during the experiment was 67.24 psi.

From Equation 20, the applied shear stress around the perimeter is derived from this measured pressure-time curve, $p(t)$, for input to WAC.

$$p_s(t) = p(t) \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)} = p(t) \frac{18.7 \cdot 7.3}{5.08 \cdot (2 \cdot 18.7 + 2 \cdot 7.3)} = 0.516 \cdot p(t)$$

The resistance function for input to WAC is determined as follows. Since the seal did not fail during Test 508, the experimental lower bound of the maximum shear-stress resistance from Equation 39 is

$$(\tau_{\text{max}})_{E.L.} = p(t)_{\text{max}} \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)} \geq 1.6 \sqrt{F_c'}$$

$$= 67.24 \text{ psi} \frac{18.7 \cdot 7.3}{5.08 \cdot (2 \cdot 18.7 + 2 \cdot 7.3)}$$

$$= 34.72 \text{ psi} \geq 1.6 \sqrt{143} = 19.13 \text{ psi}$$

The experimental lower bound maximum shear-stress resistance from Equation 36 is equal to

$$(\tau_{\text{max}})_{E.L.} = 34.72 \text{ psi} \geq 19.13 \text{ psi}$$
Therefore, the new lower bound estimate for maximum shear-stress resistance based on experimental testing is

\[(\tau_{\text{max}})_L = 34.72 \text{ psi}\]

The upper (U) bound maximum shear-stress resistance from Equation 37 is equal to

\[(\tau_{\text{max}})_U = 6\sqrt{F'_e} = 6\sqrt{143} = 71.75 \text{ psi}\]

For the purpose of this SDOF analysis, it is assumed that the maximum shear-stress resistance is equal to the lower bound maximum shear-stress resistance.
The shear-stress resistance in Table 21 is calculated from the normalized shear resistance in Table 19 through multiplication by the maximum shear resistance, \( \tau_{\text{max}} \), of 34.72 psi. Figure 36 shows this shear resistance function, which was input into the WAC for an SDOF analysis.

The effective density for input into WAC is calculated using Equation 30.

\[
\gamma' = \gamma \frac{W \cdot H}{W' \cdot H'} = 48.77 \text{ pcf} \frac{18.7 \cdot 7.3}{16.26 \cdot 16.26} = 25.18 \text{ pcf}
\]

Figure 37 shows a comparison of the measured displacement-time history (shown in red) from Test 508 to the calculated displacement-time history (shown in blue) from the SDOF analysis using the shear-stress resistance function in the WAC. The predicted maximum displacement from the SDOF analysis is higher than that in the experimental, meaning that the actual shear-stress resistance is greater than the lower bound maximum shear stress of 34.72 psi that was selected for this analysis.
Figure 36. Shear-stress resistance function for Test 508.

Figure 37. Measured displacement-time history (in red) from Test 508 compared to the calculated displacement-time history (in blue) from the SDOF analysis.
4.1.3.2 Analysis of Test 509

Test 509 used the same structure as Test 508, but this test developed a much higher frontal pressure. Figure 38 shows the frontal pressure-versus-time data obtained during test 509. The maximum frontal pressure, \( p_{\text{max}} \), during the experiment was 191.8 psi.

The applied shear stress around the seal perimeter for input to WAC was calculated using Equation 20 in the same manner as done for Test 508.

\[
p_s(t) = 0.516 p(t)
\]

A modified resistance function for input to WAC was determined as follows. Since the seal failed during Test 509, the experimental upper bound of the maximum shear-stress resistance from Equation 40 is equal to

\[
(\tau_{\text{max}})_{E.U.} = p(t)_{\text{max}} \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)} \leq 6 \sqrt{F_c'}
\]
\[(\tau_{max})_{E.U.} = P_{max} \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)} = 191.8 \text{ psi} \cdot 0.516 \]
\[= 98.97 \text{ psi} \leq 6\sqrt{143} = 71.75 \text{ psi} \]

Therefore, the upper bound maximum shear stress is equal to

\[(\tau_{max})_U = 71.75 \text{ psi} \]

From Test 508 it is known that the lower bound maximum shear stress was greater than 34.72 psi. Therefore, the range for the maximum shear stress is

\[34.72 \text{ psi} \leq \tau_{max} \leq 71.75 \text{ psi} \]

For this SDOF analysis, an average value of 53.24 psi was used for the maximum shear stress. The shear-stress resistance in Table 22 is calculated from the normalized shear resistance in Table 19 through multiplication by the maximum shear resistance, \(\tau_{max}\), of 53.24 psi. Figure 39 shows the new shear resistance function that was input into WAC for SDOF analysis.

The equivalent set of seal dimensions for this WAC analysis, \(W'\) and \(H'\), are the same as before, i.e., 16.26 ft. Similarly, the effective density, \(\gamma'\), is the same, i.e., 25.18 pcf.

<table>
<thead>
<tr>
<th>Shear Displacement, in</th>
<th>Shear Resistance, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.0034</td>
<td>5.324</td>
</tr>
<tr>
<td>0.3215</td>
<td>44.72</td>
</tr>
<tr>
<td>0.8113</td>
<td>51.64</td>
</tr>
<tr>
<td>1.2400</td>
<td>53.24</td>
</tr>
<tr>
<td>4.1500</td>
<td>52.17</td>
</tr>
<tr>
<td>9.3300</td>
<td>40.99</td>
</tr>
</tbody>
</table>
Figure 40 compares the measured displacement-time history (in red) from Test 509 to the calculated displacement-time history (in blue) from the SDOF analysis using the modified shear-stress resistance function in WAC. The agreement between the calculation and the experiment is excellent, which provides validation of the SDOF analysis method with a shear resistance function, provided that reliable estimates of the maximum shear stress are available.

4.1.4 Analysis of cement foam plug seal to meet 120-psi design pressure-time curve using the equivalent static analysis method

This section uses relationships for estimating bounds on the shear strength of cement foam to calculate the seal thickness required to resist the current MSHA 120-psi pressure-time curve with instantaneous rise time for coal mine seal design. The analysis considers a 20-ft-wide-by-7-ft-high entry and cement foam with compressive strength of 350 psi. The equivalent static analysis method is used in this example.

From Section 3.2, the dynamic load factor (DLF) for the design pressure-time curve is 2. The equivalent frontal pressure is calculated as

\[ p_e = p \cdot DLF = 120 \cdot 2 = 240 \text{ psi} \]
The equivalent frontal load is therefore

\[ p_e = p_e \cdot W \cdot H = 240 \cdot 20 \cdot 12 \cdot 7 \cdot 12 = 4,838,400 \text{ lb} \]

Equations 36 and 37 estimate the lower and upper bounds for the shear strength of cement foam based on its compressive strength.

\[ (\tau_{\text{max}})_L = 1.6\sqrt{F'_c} = 1.6\sqrt{350} = 29.9 \text{ psi} \]

\[ (\tau_{\text{max}})_U = 6\sqrt{F'_c} = 6\sqrt{350} = 112.3 \text{ psi} \]

The lower and upper bound total shear resistances at the interface are therefore

\[ V_L = (\tau_{\text{max}})_L \cdot (2 \cdot W + 2 \cdot H) \cdot T = 29.9 \cdot (2 \cdot 20 \cdot 12 + 2 \cdot 7 \cdot 12) \cdot T_L = 19,375 \cdot T_L \]

\[ V_U = (\tau_{\text{max}})_L \cdot (2 \cdot W + 2 \cdot H) \cdot T = 112.3 \cdot (2 \cdot 20 \cdot 12 + 2 \cdot 7 \cdot 12) \cdot T_U = 72,770 \cdot T_U \]
The total shear resistance must exceed the frontal load. The design relationship is therefore

\[ V \geq P_e \]

Substituting the lower and upper bound shear resistances and solving for \( T \) results in

\[ T_L \geq \frac{4,838,400}{19,375} = 250 \text{ in.} = 20 \text{ ft 10 in.} \]

\[ T_U \geq \frac{4,838,400}{72,770} = 67 \text{ in.} = 5 \text{ ft 7 in.} \]

The range for the minimum seal thickness required to resist the 120-psi, instantaneous rise-time pressure-time curve is 5 ft 7 in. to 20 ft 10 in., depending on the shear strength of the cement foam. Note that this analysis does not include any safety factors or a dynamic increase factor for the strength of the cement foam.

The lower and upper bound estimates for \( \tau_{\text{max}} \) are estimates of the ultimate maximum shear strength for the generic shear-stress resistance function presented in Table 19. However, satisfying the “elasticity of design” criterion in the Final Rule (2008) would require use of the elastic maximum shear strength and not the ultimate maximum shear strength. For the generic shear-stress resistance function given in Table 19, the elastic maximum shear strength is 84% of the ultimate maximum shear strength. For the shear resistance function shown in Figure 32 for 350-psi cement foam, the elastic maximum shear stress is \( 0.84 \times 35.93 = 30.28 \text{ psi} \), and seal designers would use this value to satisfy the “elasticity of design” requirement.

Applying the 84% factor to the lower and upper bound estimates for the ultimate shear strength determined above gives

\[ (t_{\text{max}})_L = 0.84 \times 29.9 \text{ psi} = 25.1 \text{ psi} \]

\[ (t_{\text{max}})_U = 0.84 \times 112.3 \text{ psi} = 94.3 \text{ psi} \]
Lower and upper bound total shear resistances at the interface are therefore

\[ V_L = 0.84 \cdot 19,375 \cdot T_L = 16,275 \cdot T_L \]

\[ V_U = 0.84 \cdot 72,770 \cdot T_U = 61,127 \cdot T_U \]

The lower and upper bound estimates for seal thickness using the elastic maximum shear strength is therefore

\[ T_L \geq \frac{4,838,400}{16,275} = 298 \text{ in.} = 24 \text{ ft 10 in.} \]

\[ T_U \geq \frac{4,838,400}{61,124} = 80 \text{ in.} = 6 \text{ ft 8 in.} \]

4.1.5 Analysis of cement foam plug seal to meet the 120-psi design pressure-time curve using WAC

Estimates of the lower and upper bound for the maximum shear strength are 29.9 to 112.3 psi for cement foam with compressive strength of 350 psi. These shear-strength bounds are used to calculate lower and upper bound shear-stress resistance functions from Table 19 that can then be used in WAC to calculate a range for the required seal thickness. The WAC analysis considers the inertial effects of the seal mass and is likely to be less conservative than the equivalent static analysis method.

Figures 41a and 41b show the calculated displacement versus time responses for two seals using the lower and upper bound shear resistance functions. An optimal seal thickness was calculated with each function. Using the lower bound shear resistance function in WAC, the required seal thickness was 248 in. (20 ft, 8 in.), which is 2 in. less than the thickness determined using the equivalent static analysis method. The small difference arises because the SDOF analysis considers the inertia of the seal mass; however, the difference is small because the design remains within the elastic range. Using the upper bound shear resistance function in WAC, the required seal thickness is 66 in. (5 ft, 6 in.), which is 1 in. less than the thickness determined using the equivalent static analysis method.
The lower and upper bound estimates for the maximum elastic shear strengths are 25.1 and 94.3 psi. Using the lower bound shear resistance function in WAC, the required seal thickness is 296 in. (24 ft, 8 in.), which is 2 in. less than the thickness determined using the equivalent static analysis method. Using the upper bound shear resistance function in WAC, the required seal thickness is 79 in. (6 ft, 7 in.), which is 1 in. less than the thickness determined using the equivalent static analysis method.

### 4.2 Analysis of polyurethane foam and aggregate plug seals

Prior to the issuance of the Final Rule (2008), polyurethane foam and coarse aggregate were mixed together to make plug seals for coal mines. Seals of this kind were tested at LLL, and from one of those tests, it is possible to develop a shear-stress resistance function for use in SDOF analyses with WAC. This section provides background information on applications of polyurethane foam and aggregate in military and civil construction and develops a shear-stress resistance function for this material based on full-scale tests.

Polyurethanes are one of the most versatile foams because they generally exhibit the highest tensile and compressive strengths of all other foam
types used in structural applications. Additionally, polyurethane can be manufactured with a wide variety of properties, ranging from very soft and flexible to very strong and rigid, through simple formulation changes. The ingredients needed to make polyurethane foams are easily obtained, transported, mixed, and formed in place. For these reasons, polyurethane foams have been the material of choice for certain military and civil construction applications (Priddy et al. 2007).

There are many new application opportunities for polyurethane materials, especially the lightweight and ultra-lightweight structural foams. These applications include, for example, the repair of concrete foundation slabs and highway and airfield pavements (Priddy et al. 2007). In addition to structural reinforcement applications, past research was conducted to determine the applicability of injecting polyurethane foam into soil as a soil stabilizer.

The use of foams as a soil backfill material was also investigated in laboratory and field investigations to determine the minimum free rise density of expanding foam (Priddy et al. 2007). Results of this work indicate that an 8- to 15-lb/ft³, free-rise-density, rigid polyurethane foam can perform satisfactorily as a backfill replacement material when placed under a rigid material such as a concrete pavement. Priddy et al. (2008) recommend that, for backfill replacement, the foam must provide a minimum unconfined compressive strength of at least 200 psi at a strain of 2%.

In an effort to reduce costs of using large volumes of foam for backfill, a foam composite system can be created by mixing aggregate filler with the foam. Although the aggregate filler might improve the overall strength of lower density foam, laboratory results indicate that the addition of aggregate does not provide a significant increase in the unconfined compressive strength of the mixture. Problems have occurred with the placement of foam composites in the field. Uncontrollable volumetric expansion in the foam composite due to the addition of aggregate particles prevented proper field placement. In further laboratory testing, it was found that, with the addition of aggregate fillers, the amount of polyurethane foam that was required increased by a factor of two when compared to the amount of neat foam needed to fill the same volume (Priddy et al. 2008).
A review of the literature by Priddy and Hodo (2009) addressed issues concerning the current usage, chemistry, analytical methods, and shortcomings for predicting the performance of polyurethane foam composites. This study concluded that extensive research had been conducted in measuring the mechanical properties of both polyurethane foams and other cellular materials, but few studies focused on closed-cell, rigid, high-density foams, and almost no information is available beyond the elastic limit. They also concluded that existing foam models assume that the materials behave elastically and do not consider the predominant effects of the nonlinear phases of deformation within the foam microstructure. In addition, current models do not adequately address the micro-mechanical response due to the addition of fillers to the foam system such as aggregate.

4.2.1 Shear-stress resistance function for plug seals constructed from polyurethane foam and aggregate

Composite plugs of polyurethane foam with aggregate were used in coal mines as seals from the late 1990s through 2006. The seals were generally constructed of a polyurethane-aggregate core sandwiched between supporting forms, such as concrete block. Figure 42 shows this basic design for a 20-psi seal. Later seals, designed for 50-psi or greater loads, used foam blocks as the formwork. The seals were constructed in lifts by placing a limestone aggregate bed and then pumping polyurethane over the aggregate. As the polyurethane rose, the aggregate was carried up with the foam, creating the aggregate and foam mixture. The height of a typical lift was 8 to 12 in. depending on the thickness of the seal. This type of seal did not have any reinforcing and was not hitched into the surrounding rock. Friction between the seal material and the surrounding rock held the seal in place.

Limited testing was conducted on this type of seal, including one hydrostatic test reported by Dolinar et al. (2008) and six explosion tests reported by Weiss et al. (1996). The hydrostatic test was used to develop an ultimate shear-stress resistance value for determining allowable pressure loading, as well as a shear-stress resistance function for evaluating this seal against the new MSHA design loads. Unfortunately, all prior explosion tests were visual, pass-fail tests without any displacement-measuring instrumentation, and therefore they cannot be used for comparison with SDOF calculations.
Data from LLL Test C8 (Dolinar et al. 2008; Zipf et al. 2009) were used to develop an approximate shear resistance function for polyurethane foam and aggregate plug seals that fail along the seal-rock interface. Test C8 was a static uniform pressure test conducted in a small hydrostatic chamber using water pressure for the load. Table 23 summarizes key dimensions and properties of this test seal. Figure 43 shows the locations of the LVDTs used to measure the seal displacements in the test. Data from the instruments at points 4 and 11 near the seal-rock interface were used to determine the shear-stress resistance function.

**Table 23. Dimensions and properties of polyurethane foam and aggregate plug seal for development of a shear-stress resistance function.**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width, ( W )</td>
<td>20.4 ft (245 in.)</td>
</tr>
<tr>
<td>Height, ( H )</td>
<td>9.0 ft (108 in.)</td>
</tr>
<tr>
<td>Total seal thickness</td>
<td>2.83 ft (34 in.)</td>
</tr>
<tr>
<td>( f'_c ) for polyurethane foam and aggregate mixture</td>
<td>35.1 psi</td>
</tr>
<tr>
<td>Polyurethane foam and aggregate density</td>
<td>33.7 pcf</td>
</tr>
<tr>
<td>Concrete block density</td>
<td>140 pcf</td>
</tr>
<tr>
<td>Average seal density</td>
<td>83.7 pcf</td>
</tr>
</tbody>
</table>
The hydrostatic pressure applied to the face of the seals rose to a peak of 17.7 psi in about 400 sec. The applied shear stress at the seal/rock interface as calculated by Equation 20 is

\[ p_s(t) = p(t) \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)} = p(t) \frac{20.4 \cdot 9.0}{2.83 \cdot (2 \cdot 20.4 + 2 \cdot 9.0)} = 1.10 p(t) \]  

(41)

This applied pressure-time curve was input to WAC to define the applied load in the SDOF analysis.

The pressure-versus-displacement data from LVDTs 4 and 11, which were located close to the seal-rock interface, are shown in Figure 44. The pressure-displacement relationship is linear elastic up to a load of about 16 psi at 0.05 in. of displacement and then becomes inelastic until the pressure reaches a maximum of 17.7 psi at 0.15 in. The pressure then decreases to about 12.9 psi in a softening region that extends out to 0.64 in. of displacement and remains constant at about 12.9 psi out to 3 in. of displacement where complete failure occurs. Table 24 summarizes the pressure-displacement data that approximate the measured data.

The shear-stress resistance at the interface around the seal perimeter is related to the applied frontal pressure by Equation 23, which gives

\[ r_s(t) = p(t) \frac{W \cdot H}{T \cdot (2 \cdot W + 2 \cdot H)} = p(t) \frac{20.4 \cdot 9.0}{2.83 \cdot (2 \cdot 20.4 + 2 \cdot 9.0)} = 1.10 p(t) \]  

(42)
Polyurethane foam and aggregate materials are not included among the list of materials in Table 17 of UFC 3-340-02 (2008), which tabulates the Dynamic Increase Factors (DIF) for design. The DIF for direct shear in conventional concrete is 1.10. Most materials exhibit an increase in strength when subjected to dynamic loads. The strength increase is not observed in the data from this test since it was a static loading test. For the
purposes of this study, a DIF of 1.10 was assumed when developing the shear-stress resistant function.

\[
s_r(t) = 1.10 \cdot p(t) \cdot DIF = 1.10 \cdot p(t) \cdot 1.10 = 1.21 \cdot p(t) \quad (43)
\]

Substituting in the equation above with the values for the frontal load obtained in Test C8, the shear resistance can be obtained. The shear strain is calculated from the measured displacements divided by the interface thickness of 2.83 ft (34 in.). Table 25 shows the shear strain and shear-stress resistance values at the seal/rock interface for Test C8, and Figure 45 shows the shear-stress resistance curve.

This shear strain resistance was developed from a single test and only applies to a polyurethane foam and aggregate plug seal where the compressive strength of the mixture is about 35 psi. This function can be input to WAC to calculate the dynamic response of similar structures made from similar materials but with different geometries and applied pressure loads.

4.2.2 Analysis of polyurethane foam and aggregate plug seal to meet the 120-psi design pressure-time curve using the equivalent static analysis method

This section uses the estimated shear strength of a polyurethane foam and aggregate mixture to calculate the seal thickness required to resist the current MSHA 120-psi, instantaneous rise time, pressure-time curve for coal mine seal design. The analysis considers a 20-ft-wide-by-7-ft-high entry. The equivalent static analysis method is used in this example.

<table>
<thead>
<tr>
<th>Shear Strain, in./in.</th>
<th>Shear-Stress Resistance, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0000</td>
<td>0.0</td>
</tr>
<tr>
<td>0.0015</td>
<td>19.0</td>
</tr>
<tr>
<td>0.0045</td>
<td>21.4</td>
</tr>
<tr>
<td>0.0190</td>
<td>15.6</td>
</tr>
<tr>
<td>0.0882</td>
<td>15.6</td>
</tr>
</tbody>
</table>
From Section 3.2 of this report, the dynamic load factor (DLF) for the design pressure-time curve is 2. The equivalent frontal pressure is calculated as

\[ p_e = p \cdot DLF = 120 \cdot 2 = 240 \text{ psi} \]

The equivalent frontal load is therefore

\[ P_e = p_e \cdot W \cdot H = 240 \cdot 20 \cdot 12 \cdot 7 \cdot 12 = 4,838,400 \text{ pounds} \]

The shear strength of the polyurethane foam and aggregate mixture is 21.4 psi, from Table 25, i.e.,

\[ (\tau_{\text{max}}) = 21.4 \text{ psi} \]

The total shear resistance at the interface is therefore
\[ V = (\tau_{max}) \cdot (2 \cdot W + 2 \cdot H) \cdot T = 21.4 \cdot (2 \cdot 20 \cdot 12 + 2 \cdot 7 \cdot 12) \cdot T = 13,867 \cdot T \]

The total shear resistance must exceed the frontal load. The design relationship is

\[ V \geq P_e \]

Substituting the shear resistance and solving for \( T \) results in

\[ T \geq \frac{4,838,400}{13,867} = 349 \text{ in.} = 29 \text{ ft 1 in.} \]

Note that this analysis does not include any safety factors or a dynamic increase factor for the strength of the polyurethane foam and aggregate mixture.

4.2.3 Analysis of a polyurethane foam and aggregate plug seal to meet the 120-psi design pressure-time curve using WAC

The resistance function given in Table 25 for a polyurethane foam and aggregate plug seal was used in WAC to determine the seal thickness required to resist the 120-psi design pressure-time curve with instantaneous rise time. Based on the maximum shear strength of 21.4 psi, the required seal thickness is 347 in. (28 ft, 11 in.), which is 2 in. less than the thickness determined using the equivalent static analysis method. If the maximum elastic shear strength of 19 psi is used in the WAC analysis, the required seal thickness is 390 in. (32 ft, 6 in.). Figure 46 shows the calculated displacement-time response for this design case.
Figure 46. Computed displacement-time response of a polyurethane-and-aggregate seal to resist a 120-psi pressure-time curve with an instantaneous rise time.

WACMS Analysis: Displacement-Time
Polyurethane Foam and Aggregate
5 Analysis of Seal Foundations

Both the seal structure and its foundation must resist the design pressure-time loads indicated by the curves in the seal regulations. The design explosion load applied to the face of a seal must be transferred through the seal structure into the surrounding coal, roof, and floor rock. This chapter considers two methods to anchor a seal, i.e., hitches and rock-bolt anchors. A hitch is a shallow excavation in the rock surrounding the seal and is designed to hold the structure in place when it is subjected to a design load. The bearing strength of the rock provides the resisting force to anchor the seal. However, the mechanical properties of the surrounding rock mass are highly variable, difficult to measure, and may be inadequate. Therefore, the recommended method to anchor a seal is through rock bolts embedded in the seal structure and extending into the surrounding rock. These steel anchors can provide sufficient shear or tensile resistance to hold the seal in place under a design load. This section presents methods to analyze the anchorage capacity of hitches and rock-bolt anchors for seal foundations.

5.1 Anchorage capacity of hitches

Two engineering methods are used to assess the anchorage capacity of a hitch, i.e., a simple analytic method and a finite element analysis. The first method uses a bearing capacity analysis to calculate the anchorage capacity of a hitch. The method depends on empirically determined estimates of rock quality and might be considered conservative. The second method, a finite element analysis, shows that the bearing strength of a hitch may be 2 to 3.5 times greater than that indicated by the first, simple method. However, as will be discussed later, there are many ways that a hitch can fail as a seal anchorage system, and for this reason, they are not the preferred method to anchor a seal.

5.1.1 Description of possible seal hitch methods in coal mines

A hitch is an excavation into the coal ribs, floor rock, and possibly the roof rock that creates a bearing surface around the seal perimeter in which to anchor the seal. If a seal designer elects to use a hitch for anchorage, the engineering question is how deep to excavate the hitch to achieve the
required anchorage capacity, which will depend on geotechnical properties of the rock surrounding the seal.

Figure 47 is a side-view of a typical hitch that could be cut into the floor rock of a coal mine. Because the hitch will likely be excavated with a continuous mining machine, it will not have a rectangular profile in the floor rock. A continuous mining machine excavates coal and rock with a rotating cutter drum about 12-ft long and 3 ft in diameter. This cylindrical cutting drum can cut a sloping profile into the floor rock as shown in Figure 47. The resulting bearing surface for the hitch will have the circular profile shown. With a continuous mining machine, it is possible to excavate 2 or 3 ft into the floor rock without difficulty, depending on the strength of the rock.

In order to preserve the integrity of the roof rock for safety against potential rock falls, a hitch into the roof rock is usually not excavated in coal mines. Hence, a roof hitch is not shown in Figure 47.

Figure 48 is a plan view of a typical hitch that could be cut into the coal ribs of a coal mine. If a continuous mining machine is used to excavate the
hitch, the likely profile in plan-view is a right triangle. A continuous mining machine can easily excavate a rib hitch to a depth of 3 or 4 ft without difficulty.

![Plan view of seal hitch cut into coal ribs with a continuous mining machine.](image)

5.1.2 Bearing capacity analysis of hitches

The explosion load on the seal face is transferred through the seal and onto the bearing face of the hitch in a coal rib, the floor rock, or possibly the roof rock. Figure 49 shows this load transfer and the rock area that is considered in this bearing analysis. The engineering manual “Engineering and Design – Rock Foundations” (EM 1110-1-2908; USACE 1994) presents a method to calculate the bearing capacity of a hitch depending on the geotechnical properties of the coal or rock. Although intended for traditional foundation design, such as a gravity dam bearing on a rock mass, the methods in this manual apply directly to the design of hitches for coal mine seals.

As shown in Figure 47 through 49, the bearing load acts parallel to bedding planes in the mine rib, floor rock, or roof rock in a coal mine seal hitch. Figure 50 shows the applicable bearing capacity failure mode for analysis. The steeply dipping joints (or bedding planes) are assumed to be open with a spacing of $S$, which is less than the bearing width $B$. These
rock columns are assumed to behave independently of one another and are assumed to fail in unconfined compression. These assumptions are very conservative.

Figure 49. Section view of coal mine seal showing idealized hitch and rock area considered for bearing analysis.

Figure 50. Bearing capacity failure mode applicable to coal mine seal hitches. (Figure 68 from USACE 1994). The bearing load of width B is parallel to open bedding planes of spacing S, with S < B. The failure mode is unconfined compressive failure of the rock columns.
The unconfined compressive strength (UCS) or bearing strength, \( q_u \), of these columns is estimated using equation 6-5 from USACE (1994).

\[
q_u = 2 \cdot c \cdot \tan\left(45 + \frac{\phi}{2}\right)
\]  

where

\[
c = \text{cohesion of the rock mass}
\]

\[
\phi = \text{angle of internal friction for the rock mass}.
\]

Equation 44 is based on the Mohr-Coulomb failure criteria. Table 26 (after Zipf 2006) presents laboratory and field values for the UCS parallel to bedding planes, along with corresponding values for cohesion, \( c \), and friction angle, \( \phi \), for a wide range of rocks commonly found in coal mines. Given \( c \) and \( \phi \) from this table, Equation 40 gives the field value of the UCS, which is identical to the bearing strength of the rock mass parallel to the bedding planes.

Table 26. Properties of coal mine rocks parallel to bedding planes and the calculated bearing strength from Equation 40.

<table>
<thead>
<tr>
<th>Material Name</th>
<th>Description</th>
<th>Lab UCS (psi)</th>
<th>Field UCS (psi)</th>
<th>Young's Modulus ( \times 10^6 ) psi</th>
<th>Cohesion (psi)</th>
<th>Friction Angle (deg)</th>
<th>Dilation Angle (deg)</th>
<th>Tensile Strength (psi)</th>
<th>Bearing Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil 1</td>
<td>Paste</td>
<td>6</td>
<td>3</td>
<td>0.145</td>
<td>1</td>
<td>21</td>
<td>10</td>
<td>0.3</td>
<td>3</td>
</tr>
<tr>
<td>Soil 2</td>
<td>Very soft soil</td>
<td>10</td>
<td>6</td>
<td>0.145</td>
<td>2</td>
<td>21</td>
<td>10</td>
<td>0.6</td>
<td>6</td>
</tr>
<tr>
<td>Soil 3</td>
<td>Soft soil</td>
<td>20</td>
<td>12</td>
<td>0.145</td>
<td>4</td>
<td>21</td>
<td>10</td>
<td>1.2</td>
<td>12</td>
</tr>
<tr>
<td>Soil 4</td>
<td>Firm soil</td>
<td>42</td>
<td>23</td>
<td>0.218</td>
<td>8</td>
<td>21</td>
<td>10</td>
<td>2.3</td>
<td>23</td>
</tr>
<tr>
<td>Soil 5</td>
<td>Stiff soil</td>
<td>91</td>
<td>51</td>
<td>0.290</td>
<td>17</td>
<td>21</td>
<td>10</td>
<td>5</td>
<td>49</td>
</tr>
<tr>
<td>Soil 6</td>
<td>Very stiff soil</td>
<td>203</td>
<td>116</td>
<td>0.363</td>
<td>39</td>
<td>21</td>
<td>10</td>
<td>12</td>
<td>113</td>
</tr>
<tr>
<td>Rock 1</td>
<td>Claystone</td>
<td>392</td>
<td>218</td>
<td>0.435</td>
<td>73</td>
<td>21</td>
<td>10</td>
<td>22</td>
<td>212</td>
</tr>
<tr>
<td>Rock 2</td>
<td>Black shale</td>
<td>783</td>
<td>435</td>
<td>0.580</td>
<td>145</td>
<td>22</td>
<td>10</td>
<td>44</td>
<td>430</td>
</tr>
<tr>
<td>Rock 3</td>
<td>Gray shale</td>
<td>1,450</td>
<td>827</td>
<td>0.725</td>
<td>276</td>
<td>23</td>
<td>10</td>
<td>87</td>
<td>834</td>
</tr>
<tr>
<td>Rock 4</td>
<td>Gray shale</td>
<td>2,610</td>
<td>1,450</td>
<td>0.870</td>
<td>479</td>
<td>24</td>
<td>10</td>
<td>145</td>
<td>1,475</td>
</tr>
<tr>
<td>Rock 5</td>
<td>Gray shale</td>
<td>3,625</td>
<td>2,030</td>
<td>1.015</td>
<td>653</td>
<td>25</td>
<td>10</td>
<td>203</td>
<td>2,050</td>
</tr>
<tr>
<td>Rock 6</td>
<td>Siltstone</td>
<td>4,640</td>
<td>2,610</td>
<td>1.160</td>
<td>798</td>
<td>26</td>
<td>10</td>
<td>247</td>
<td>2,554</td>
</tr>
<tr>
<td>Rock 7</td>
<td>Siltstone</td>
<td>5,945</td>
<td>3,335</td>
<td>1.450</td>
<td>1,015</td>
<td>27</td>
<td>10</td>
<td>334</td>
<td>3,313</td>
</tr>
<tr>
<td>Rock 8</td>
<td>Sandstone</td>
<td>8,555</td>
<td>4,785</td>
<td>1.740</td>
<td>1,450</td>
<td>28</td>
<td>10</td>
<td>479</td>
<td>4,826</td>
</tr>
<tr>
<td>Rock 9</td>
<td>Sandstone</td>
<td>12,470</td>
<td>6,960</td>
<td>2.175</td>
<td>2,030</td>
<td>29</td>
<td>10</td>
<td>696</td>
<td>6,893</td>
</tr>
</tbody>
</table>
### Table 1: Material Properties

<table>
<thead>
<tr>
<th>Material Name</th>
<th>Description</th>
<th>Lab UCS (psi)</th>
<th>Field UCS (psi)</th>
<th>Young’s Modulus ($\times 10^6$ psi)</th>
<th>Cohesion (psi)</th>
<th>Friction Angle (deg)</th>
<th>Dilation Angle (deg)</th>
<th>Tensile Strength (psi)</th>
<th>Bearing Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock 10</td>
<td>Limestone</td>
<td>17,835</td>
<td>10,005</td>
<td>2.900</td>
<td>2,900</td>
<td>30</td>
<td>10</td>
<td>986</td>
<td>10,046</td>
</tr>
<tr>
<td>Coal 1</td>
<td>Bright coal</td>
<td>232</td>
<td>131</td>
<td>0.363</td>
<td>44</td>
<td>25</td>
<td>10</td>
<td>12</td>
<td>138</td>
</tr>
<tr>
<td>Coal 2</td>
<td>Banded coal</td>
<td>421</td>
<td>232</td>
<td>0.363</td>
<td>73</td>
<td>26</td>
<td>10</td>
<td>22</td>
<td>234</td>
</tr>
<tr>
<td>Coal 3</td>
<td>Dull coal</td>
<td>928</td>
<td>522</td>
<td>0.363</td>
<td>160</td>
<td>27</td>
<td>10</td>
<td>44</td>
<td>522</td>
</tr>
<tr>
<td>Coal 4</td>
<td>Dull coal</td>
<td>1,740</td>
<td>972</td>
<td>0.363</td>
<td>290</td>
<td>28</td>
<td>10</td>
<td>87</td>
<td>965</td>
</tr>
</tbody>
</table>

#### 5.1.3 Simple design analysis of a hitch

The following example illustrates the design of a hitch to resist the 120-psi instantaneous rise time, pressure-time curve for coal mine seals. The example seal is in an entry measuring 20-ft wide by 7-ft high. The coal along the ribs of this entry is assumed to be Coal 3 in Table 26 with a bearing strength of 522 psi. The floor rock is assumed to be black shale with a bearing strength of 430 psi. This seal is not hitched into the roof rock.

The Dynamic Load Factor (DLF) for this pressure-time curve is 2, as discussed earlier in Chapter 3 of this report. Therefore, the seal load is

$$\text{Seal Load} = p(t) \cdot W \cdot H \cdot DLF = 120 \cdot 20 \cdot 12 \cdot 7 \cdot 12 \cdot 2 = 4,838,400 \text{ lb}$$

This seal load must be resisted by the bearing capacity of the seal hitch. Assume that a hitch 3-ft deep is excavated into each coal rib and a 2-ft-deep hitch into the black shale floor. The bearing capacities of the rib and floor are

Bearing capacity of rib = $2 \cdot \text{bearing strength} \cdot H \cdot D_r$

$$= 2 \cdot 522 \cdot 7 \cdot 12 \cdot 3 \cdot 12 = 3,157,056 \text{ lb}$$

Bearing capacity of floor = $\text{bearing strength} \cdot H \cdot D_f$

$$= 430 \cdot 20 \cdot 12 \cdot 2 \cdot 12 = 2,476,800 \text{ lb}$$

Total bearing capacity = $3,157,056 + 2,104,704 = 5,633,856 \text{ lb}$

Total bearing capacity > Seal load - OK.

This simple analytic bearing analysis method assumes that the layers of rock between bedding planes in the coal rib and floor rock act indepen-
dently of one another and fail in uniaxial compression. This assumption is very conservative and results in a very conservative design for a seal hitch.

5.1.4 Finite element analysis of hitches

A finite element analysis was performed to quantify approximately how conservative is the analytic bearing analysis. The finite element program HONDO (Key et al. 1978) was used with the constitutive model described by Merkel and Dass (1985). The analysis considers a cross section of the floor rock or coal rib material subjected to a bearing load that is derived from the explosion load as shown in Figure 51 (right). The finite element model is rotated 90 deg as shown in Figure 51 (left). The depth of the hitch is the width of the applied bearing load. Displacement boundary conditions on the model are as indicated in Figure 51 (left). The boundary conditions along \( Y = 0 \) and along \( X = 40 \) are far away from the applied bearing load and are not affected by it. The bearing load is applied as a uniform bearing stress and is increased at a constant rate. Two different models are considered, i.e., one with a hitch depth (bearing load width) of 12 in., and the other with a depth of 24 in. The average \( Y \) displacement under the bearing load along \( Y = 100 \) is also recorded.

Figure 51. Plane-strain finite element model of rock comprising seal hitch (left) and relationship of the model to the entire seal structure with hitch (right).
Figure 52 shows plots of the deformed finite element grid and the effective plastic strain in the elements for the 12- and 24-in.-deep hitch depths. As the applied bearing load across the boundary at Y = 100 increases, the effective plastic strain in the elements under the load increases. These plots show the loading stage when the zone of an effective plastic strain (EPS) value of 1.0 extends from the deepest part of the hitch to the free surface along X = 0. This zone corresponds to the classic shear failure plane that one expects to develop in this loading case. For both cases, the bearing stress at failure is about 1,500 psi at an average Y displacement under the load of about 8 to 10 in.

Figure 53 is a plot of the applied bearing stress versus the average Y displacement under the bearing load along Y = 100. For both models, a 12-in.-deep hitch and a 24-in.-deep hitch, the bearing stress versus displacement is almost coincident, meaning that the model calculations are independent of the model geometry and the depth of hitch. In both cases, the bearing stress versus displacement curve departs from linear elastic at a stress of about 1,000 psi at a displacement of 1.5 in. Maximum bearing stress is assumed to occur when the EPS zone of 1 extends completely under the hitch. For this condition, the maximum bearing stress occurs at about 1,500 psi and 8 to 10 in. of displacement.

These calculations do not imply that the ultimate bearing load is independent of the hitch depth. The calculations show that the bearing stress is independent of hitch depth; however, the maximum bearing load does depend on the hitch depth. Thus, the 24-in.-deep hitch is able to resist twice as much load as the 12-in.-deep hitch, even though the bearing stresses are the same.

The material properties used in this finite element analysis correspond closest to Rock #2 in Table 26, also called “black shale.” Prior calculations using Equation 50 gave a value of 430 psi as the bearing strength. This strength corresponds to the unconfined compressive strength of the rock parallel to the bedding planes. The finite element analysis indicated that the displacement became inelastic at a bearing stress of about 1,000 psi, and suggested a bearing strength of about 1,500 psi. Thus, the “no confinement” bearing stress formula given by Equation 40 is conservative by a factor of at least 2.3, and possibly as high as 3.5.
Figure 52. Deformed finite element grid and effective plastic strain (EPS) for 12-in.-deep hitch (top) and 24-in.-deep hitch (bottom). Shear failure occurs when the EPS value of 1.0 extends from maximum hitch depths near $X = 12$ or $X = 24$ and $Y = 100$ to the free surface along $X = 0$. 
5.1.5 WAC analysis of seal foundations

The Wall Analysis Code (WAC; Slawson 1995) discussed in Chapter 3 will also calculate the dynamic support reactions at the perimeter of a seal, which can then be used to design an appropriate foundation for the seal. This example considers a reinforced concrete seal hitched into the floor rock as shown in Figure 47 and also into the coal ribs as shown in Figure 48. The design parameters are as follows.

- Concrete compressive strength – 5,000 psi
- Reinforcing steel yield strength – 60,000 psi
- Entry width – 16 ft
- Entry height – 4 ft
- Design pressure-time curve – 120 psi with instantaneous rise time
- Hitch depth (ribs and floor) – 3 ft
The analysis with WAC considers a two-way wall in which flexure is allowed in both the horizontal and vertical directions. The analysis assumes that the wall is simply supported on three sides, i.e., the floor and the two ribs. These sides have zero moment connections that are free to rotate. The top side or roof is unsupported and free to translate. This situation in the WAC is Case 34, corresponding to a two-way wall with three sides simply supported.

Analysis with the WAC arrived at a seal design in which the concrete is 36 in. thick and contains No. 10 reinforcing bars placed horizontally (rib to rib) on 4-in. centers near both faces. Figure 54 shows the computed displacement response for this structure. The structure remains within the elastic regime. The maximum dynamic displacement of 0.323 in. is less than the elastic limit of 0.340 in.

Figure 54. Computed displacement response of 36-in.-thick reinforced concrete seal, simply supported on three sides and free on the fourth side.
Figures 55 and 56 show the computed dynamic reactions along the floor and along the ribs. The maximum shear along the floor is 18,411 lb/in., and the maximum shear along the ribs is 16,281 lb/in. These dynamic reactions provide a basis for specifying a minimum hitch depth that depends on the strength of the foundation rock. The bearing pressure is equal to the maximum shear force per inch along the rib or floor, divided by the hitch depth. This bearing pressure is an average applied pressure from the top to the bottom of the hitch.

If the floor hitch is excavated 36 in. deep, the average bearing pressure in the floor rock is 18,411/36 = 511 psi. According to Table 26, any floor rock stronger than gray shale (Rock 3) has sufficient bearing strength to resist the dynamic reaction force. If the rib hitch is also excavated 36 in. deep, the average bearing pressure in the rib coal is 452 psi. As shown in Figure 57, this bearing force must be resolved perpendicular to the hitch; thus, the bearing stress becomes 452/cos 45 deg = 640 psi. According to Table 26, Coal 4 has sufficient bearing strength to resist this dynamic reaction force.

5.1.6 Summary of anchorage capacity of hitches

Hitches may have appeal as a practical, economical method to anchor coal mine seals; however, they have many drawbacks that should preclude their use as seal foundations. First, the design anchorage capacity for the hitch depends on correct assessment of the strength of the rock and coal comprising the hitch. This strength is likely to be highly variable and very difficult to measure directly. The interface bearing and shear resistances should only be used in engineering design when experimental data for the interface properties are available.

The other major problem with hitches is the possibility of fractures developing in the surrounding rock and coal that make the anchorage capacity of the hitch very low. Vertical fractures may develop in the coal ribs due to vertical stress concentrations, and horizontal fractures (floor heave) can develop in the floor rock due to horizontal stress concentrations. These fractures are parallel to the opening and may be located 1 to 5 ft or more from its edge, depending on the magnitude of the stress concentration.
The fractures can develop due to the presence of the coal mine entry alone, and any nearby full-extraction mining can only exacerbate the problem. Development of these fractures parallel to the opening can make the anchorage capacity of the hitch zero, unless the hitch is much deeper than the fractured zone.

To summarize, the design of a seal foundation using a hitch depends on a correct assessment of the rock strength and the maintenance of that strength throughout the life of the seal. These conditions may not be practically achievable. Fortunately, excellent engineering alternatives to hitches are available to anchor coal mine seals to the surrounding rock.
5.2 Anchorage capacity of rock-bolt anchors

Rock-bolt anchors are the preferred method for anchoring a coal mine seal to the surrounding rock. The steel rock-bolt anchors provide engineered shear resistance at the interface. Unlike hitches, the anchorage capacity of rock-bolt anchors is well-understood, and a seal foundation using rock-bolt anchors can be engineered and tested for adequacy.

5.2.1 Description of rock bolts and rock-bolt anchors

Rock bolts are used throughout the mining industry for ground support. Figure 58 shows one kind of rock bolt that uses slow- and fast-setting resin cartridges to anchor the bolt. Slow-setting resin cartridges are placed
behind a fast-setting resin cartridge, and the rock bolt is pushed through the cartridges and rotated to thoroughly mix the resin and its catalyst. The bolt is tensioned after the fast-setting resin anchor has set. The slow-setting resin sets later to fully grout the rock bolt in place. For mining applications, typical rock bolts are 5/8 to 1 in. in diameter (#5 through #8 bar) with lengths typically ranging from 4 to 8 ft. Bolt diameters up to 1-3/8 in.
(11 bar) and longer lengths are also readily available. Rock bolts can be anchored into the rock with resin cartridges, mechanical anchors, or cement grout. The rock bolt may be threaded on the exposed end and fitted with a nut to enable tensioning of the bolt after installation. In recent decades, cable bolts have replaced conventional rock bolts made from steel bars in certain applications. For practical considerations, the minimum spacing for rock bolts is 12 in. Closer spacing could cause issues when drilling holes in the rock or when inserting the rock bolts. Figure 59 shows the concept of using rock bolts to anchor a seal to the surrounding rock.

![Figure 59. Conceptual seal foundation design using rock bolts.](image)

The seal foundation must develop sufficient shear resistance to counteract the shear force generated by the explosion load acting on the seal structure. When using rock-bolt anchors for a seal foundation, it is desirable that the bolts act in pure shear in order to maximize their shear resistance capacity. Axial forces acting on rock-bolt anchors can significantly reduce their capacity to resist shear.

Axial forces can develop in any seal design due to flexure of the seal structure. The flexural behavior is most pronounced in reinforced concrete walls when the thickness-to-height ratio is less than 1/4. Unreinforced concrete plug seals, which generally have a thickness-to-height ratio greater than or equal to 1, will also flex and induce axial forces in rock-bolt anchors.
A finite element analysis was performed using ABAQUS (SIMULIA 2005) to investigate the axial and shear forces acting in rock-bolt anchors used as the foundation for an unreinforced concrete plug seal. This analysis provided insight into (1) the failure mode of the concrete plug seal (flexure, shear, or a combination), (2) the load transfer mechanism from the seal structure to the seal foundation, and (3) the force distribution in the rock-bolt anchors at different distances from the applied pressure loading.

The axial forces in a rock-bolt anchor are important because they can reduce the shear capacity of the anchor. The magnitude of the axial forces, hence the magnitude of the shear capacity reduction, will depend on the seal thickness and the amount of flexure developed in the seal structure. However, there are simple practical techniques available to minimize the axial forces in the rock-bolt anchors and maximize their shear resistance capacity by partially de-bonding the rock-bolt anchor.

5.2.2 Analysis of forces in rock-bolt anchors with ABAQUS

The concrete seal with rock-bolt anchors is modeled using ABAQUS. As shown in Figure 60, the basic seal model is a two-dimensional, vertical cross section measuring 84 in. high by 84 in. thick with rock-bolt anchors spaced 12 in. center-to-center. A uniform pressure of 288 psi is applied to the left side of the model. This static pressure is equivalent to the 120-psi design pressure-time curve with instantaneous rise time using a dynamic load factor of 2 and a safety factor of 1.2.

Figure 60 also shows the concrete seal separated into upper and lower pieces by a horizontal crack that is modeled as a frictionless contact surface. Each block piece measures 42 in. high by 84 in. thick. The artificial crack between the concrete blocks ensures that the concrete has zero tensile strength and can only resist compressive forces. A partition was created in the concrete at each rock-bolt anchor location to provide a permanent node connecting the rock-bolt anchors to the concrete.

The mesh of the concrete seal was created with quadratic, triangular, plane strain, modified formulation elements (CPE6M) as shown in Figure 61. These elements were chosen because of their accuracy with contact problems and their suitability for bending problems. The depth of these plain strain elements in the z direction is always 1.0. The concrete is linear elastic with a Young’s Modulus of 4,030,510 psi and a Poisson’s Ratio of 0.2. The concrete is allowed to deform without failure in these models.
Figure 60. ABAQUS model of concrete seal with rock-bolt anchors and applied load.

Figure 61. ABAQUS model of concrete seal with rock-bolt anchors showing the mesh containing 504 elements with 996 nodes.
The 48-in.-long rock-bolt anchors are modeled in ABAQUS with linear elastic beam elements as shown in Figure 60. Each of the 14 steel rock-bolt anchors in the model is discretized into eight of these linear-elastic beam elements. The steel has a Young’s Modulus of 29,000,000 psi and a Poisson’s Ratio of 0.33. The steel is also allowed to deform without failure in these models.

The rock-bolt anchors used for the foundation are restrained in the x-direction along their entire length. At the end of the rock-bolt anchors farthest from the concrete seal, no displacement or rotation is allowed. At the interface of the rock-bolt anchors and the concrete seal, the anchors are constrained so that the horizontal and vertical displacements of the last node on the bolt are coupled to the corresponding nodes in the concrete seal. The concrete is bonded to the rock-bolt anchors and causes them to stretch or contract in the vertical direction as the concrete pushes or pulls on those connections. This behavior is an important aspect of the model, as it allows the load to redistribute in response to any rotation of the concrete seal that exerts axial load on the rock-bolt anchors.

Three mesh patterns, with mesh sizes of 432, 764, and 996 nodes, were modeled in ABAQUS to check for convergence. The results of the mesh refinement study are shown in Figure 62. It was found that there was little difference between the three cases with the results only varying slightly.

As shown in Figure 63, the maximum compression stress is in the outer fiber parallel to the loaded face on the left side of the model. The stress $s_{yy}$ ($S_{22}$ in ABAQUS) gives the compression stress in the concrete, which can be compared to its yield stress. Near the centerline crack on the compression face of the concrete seal, the maximum compression stress is 394 psi. This is well below the 5000-psi yield strength of the concrete. A low amount of stress is expected since the concrete member is so thick.

The calculated shear and axial reaction forces in the rock-bolt anchors are shown in Figures 64 and 65, respectively. These calculated forces occur at the interface between the rock-bolt anchors and the concrete seal.

Table 27 summarizes the shear and axial reaction forces at this interface. As expected, the greatest shear reaction force is located close to the loading face (left side of model), and it decreases further away from this face. The axial reaction force is tensile closest to the loading face (-542 lb) and becomes more compressive (up to 1,600 lb) farther away.
Figure 62. Computed stress and deflection versus the number of nodes in the ABAQUS model for the mesh refinement study.

Figure 63. Stress contours (in psi) for stress on the y-face, in the y-direction ($s_{yy}$ or $S_{22}$) for a mesh with 996 nodes.
Figure 64. Calculated shear reaction forces (lb) in the rock-bolt anchors.

Figure 65. Calculated axial reaction forces (lb) in the rock-bolt anchors.

Table 27. Analysis of axial and shear loads on rock-bolt anchors using AISC Combined Load Factor for anchor loads and von Mises criterion for anchor stresses.

<table>
<thead>
<tr>
<th>Anchor Number</th>
<th>Axial Load (lb)</th>
<th>Shear Load (lb)</th>
<th>$F_c$ – Combined Load Factor ≤ 1</th>
<th>$F_{	ext{vM}}$ – von Mises Stress ≤ $f_{\text{dy}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-542</td>
<td>2,450</td>
<td>0.651</td>
<td>51,370 psi</td>
</tr>
<tr>
<td>2</td>
<td>81.6</td>
<td>2,430</td>
<td>0.558</td>
<td>50,550 psi</td>
</tr>
<tr>
<td>3</td>
<td>385</td>
<td>2,180</td>
<td>0.507</td>
<td>45,576 psi</td>
</tr>
<tr>
<td>4</td>
<td>611</td>
<td>1,830</td>
<td>0.419</td>
<td>38,762 psi</td>
</tr>
<tr>
<td>5</td>
<td>851</td>
<td>1,470</td>
<td>0.354</td>
<td>32,236 psi</td>
</tr>
<tr>
<td>6</td>
<td>1,116</td>
<td>1,120</td>
<td>0.318</td>
<td>26,874 psi</td>
</tr>
<tr>
<td>7</td>
<td>1,600</td>
<td>613</td>
<td>0.326</td>
<td>23,058 psi</td>
</tr>
</tbody>
</table>

In the model, the rock-bolt anchors are restrained in the x-direction along their entire length to simulate embedment in the surrounding rock, and therefore the calculated stresses in the rock-bolt anchors are not accurate along the embedded length of the anchor. Failure criteria within ABAQUS should not be used in the model elements comprising the rock-bolt anchors. However, the calculated shear and axial force reactions at the interface are accurate, and these forces can be analyzed using the AISC Combined Load Equation for the rock-bolt anchor loads (AISC 2005) or a von Mises criterion for the rock-bolt anchor stresses at the interface. Each rock-bolt
anchor must be analyzed individually since the shear and axial loading varies from anchor to anchor.

The Combined Load Factor for rock-bolt anchor loads (AISC 2005, Equation H3-6) is

\[ F_c = \left( \frac{|P_a| \cdot S_{rba}}{f_y \cdot A} \right) + \left( \frac{V \cdot S_{rba}}{0.6 \cdot f_y \cdot A} \right)^2 \leq 1.0 \]  

(45)

and the von Mises criterion for rock-bolt anchor stresses is

\[ F_{vM} = \sqrt{3} \cdot \frac{2}{6} \left[ \left( \frac{P_a \cdot S_{rba}}{A} \right)^2 + \left( \frac{V \cdot S_{rba}}{A} \right)^2 \right] \leq f_{dy} \]  

(46)

where

- \( P_a \) = axial load
- \( V \) = shear load
- \( S_{rba} \) = spacing of rock-bolt anchors
- \( f_{dy} \) = dynamic yield strength of steel
- \( f_y \) = static yield strength of steel
- \( A \) = area of rock-bolt anchors

In applying these criteria, the spacing of the rock-bolt anchors \((S_{rba})\) is 12 in.; the diameter of the anchors is 1.128 in. (Number 9 bar); their area \((A)\) is 0.999 in.\(^2\), and the yield strength of the steel is 60,000 psi. A Dynamic Increase Factor (DIF) of 1.1 is applied to this yield strength to give a dynamic yield strength \((f_{dy})\) of 66,000 psi. (See Chapter 3 for further justification of this DIF.)

As shown in Table 27, the Combined Load Factor from Equation 45 is always less than 1, and the von Mises Stress from Equation 46 is always less than the dynamic yield strength, \(f_{dy}\), of 66,000 psi. Therefore, this seal foundation design using rock-bolt anchors resists the 120-psi design pressure-time curve with an instantaneous rise time.

As mentioned earlier, a problem may exist when using rock-bolt anchors to provide shear resistance in a seal foundation. As seen in Equations 45 and 46, the presence of any axial forces in the anchors will reduce their
capacity to resist shear forces. As shown in this analysis of a seal foundation, the axial forces and the shear capacity reduction are significant even in a thick, plug-type seal where flexural forces in the seal structure are relatively small. The following section presents a simple practical technique to reduce the axial forces in the rock-bolt anchors through partial de-bonding.

### 5.2.3 Increasing shear capacity of rock-bolt anchors through local de-bonding

In reinforced concrete panels, the tensile reinforcing bars are oriented parallel to the long axis of the panel and close to its outer fiber. These bars are bonded to the surrounding concrete through mechanical means, such as friction with bar deformities, or by chemical means, such as adhesion between the mortar paste and the bar steel. The reinforcing bar is normally bonded to the concrete along its full length.

Experiments were conducted by Trasborg et al. (2012) to determine the effect of de-bonding the reinforcing bars from the concrete at the midpoint of test panels where the highest bending moment is expected. The test panels measured 48 in. long by 12 in. wide by 3 in. thick and contained two reinforcing bars of 3/8 in. diameter (Number 3 bar). Four kinds of test panels were fabricated. In the control group of panels, the reinforcing bars were bonded to the concrete along their full length. In three experimental groups of panels, the reinforcing bars were de-bonded from the concrete in the middle of the panel for lengths of 7.5, 15, and 22.5 in., which correspond to about 15, 30, and 45% of the panel length. De-bonding the reinforcing bar from the concrete was accomplished by placing thin Teflon tubes with a thickness of 1/32 in. over the reinforcing bars prior to placing the concrete, as shown in Figure 66. After the concrete cured, the test panels were loaded at the midpoint in three-point bending.

Figure 67 shows data from the control tests and the three experimental test groups. In this figure, the resistance of the panel is related to the applied load on the panel and its dimensions, and the support rotation is related to the downward displacement of the panel divided by the panel length. The control panel with full bonding of the reinforcing bar attained the highest peak resistance (or load), but then exhibited softening behavior prior to failing by fracture of the reinforcing bar at a rotation angle of about 7 deg. De-bonding the reinforcing bar led to a decrease in the panel resistance (or load-carrying capacity) of 7% to 21%; however, the panels
with de-bonded reinforcing bar did not exhibit the softening behavior, and the maximum measured support rotations increased to 13, 19, and at least 25 deg prior to failure by fracture of the reinforcing bar.

Figure 66. De-bonding longitudinal reinforcement bars from concrete by placing thin Teflon tubes over the bars (Trasborg, Olmati, and Naito 2012).
De-bonding the reinforcing bar at the midpoint of a panel, where bending moment is at a maximum, leads to a slight decrease in the load-carrying capacity of the panel but a large increase in the amount of deformation that the panel can withstand prior to failing by fracture. To enable more deformation capacity of the rock-bolt anchors used to anchor seal structures, they too should be de-bonded from the seal structure material and the surrounding rock for about 12 in. on either side of the rock seal interface. This de-bonding will minimize the shear component in the rock-bolt anchor near the rock-seal interface and maximize the axial load-carrying capacity of the anchor.
6 Analysis of Seal Structures

This chapter discusses how seal structures can fail and then presents detailed analyses for three kinds of seal structures, i.e., reinforced concrete walls, concrete plugs, and plugs made from mine waste rock or gob. The various failure modes for a reinforced concrete wall are described including flexure of the wall, diagonal shear in the wall, and shear along the foundation interface. A reinforced concrete wall analysis is presented that follows methods presented in UFC 3-340-02 (DOD 2008). An explosion test on a reinforced concrete wall is also described. The wall is analyzed using the WAC, and the calculated response of the wall is compared to the measured response from the full-scale dynamic test. The comparison demonstrates that analyses using SDOF tools such as WAC are very accurate for most practical engineering purposes. A thick concrete plug anchored to the surrounding rock with rock bolts is analyzed with the FEM. The analyses showed the stress magnitudes within the concrete plug and the combined shear and tensile loads on the rock-bolt anchors. Finally, a gob plug seal is analyzed to show the expected stresses and anchorage requirements for this unique structure.

6.1 Analysis of reinforced concrete seals

Reinforced concrete is and has been the material of choice for protective structure design in the defense community. The engineering properties of concrete and steel are well known and well understood. Concrete and steel reinforcement are inexpensive and readily available. Quality control for these materials is straightforward. For these reasons, reinforced concrete is the recommended material for coal mine seals.

The use of reinforced concrete for coal mine seals has not been widely adopted by mine operators, partly because they can be a more costly alternative to other approved designs. Reinforced concrete walls have been used as the predominant protective system for blast effects by the United States and other countries for many years. Examples include ammunition storage bunkers, aircraft hangers, blast doors, containment structures for explosives manufacturing, as well as control structures in the petrochemical industry. Unfortunately, there are no examples from NIOSH or MSHA testing to provide a basis for analyzing reinforced concrete seals in a mining environment. Regardless, more is known about how reinforced
concrete behaves under blast loading than any other material that could be used to build a mine seal.

6.1.1  Failure modes of reinforced concrete seals

A wall or seal constructed from reinforced concrete and subject to the design blast loads might fail by one of three modes, i.e., flexure, diagonal shear, or direct shear, or some combination of these modes. Analysis and design must consider all possible failure modes.

Flexural failure, shown in Figure 68, develops when either compressive or tensile bending stresses exceed some limit. The wall might fail by tensile failure of the reinforcing steel or compressive failure of the concrete and steel reinforcement combination. For the flexural failure mode to develop, the height-to-thickness ratio for the wall is generally less than 0.25.

Diagonal shear failure near the wall reactions, shown in Figure 69, is another failure mode requiring design consideration. In addition to bending stresses, the applied blast loads also induce shear stresses in the wall. The wall must have a sufficient thickness of concrete or sufficient reinforcing steel to resist these shear loads. (Note that the design of shear reinforcement for a reinforced concrete wall is not to be confused with the foundation design for the wall, or the design of very thick concrete plug seals.)
Direct shear failure through the wall itself, shown in Figure 70, is the last failure mode requiring design consideration. Direct shear is a technical term that describes a localized shear response of a structural concrete element. It can appear at areas of geometric discontinuity or load discontinuity. Extreme loads on short, deep members may result in a direct shear failure before the development of a flexural mode of response. This direct shear response is typically associated with a lower ductility or a brittle behavior mode (from Krauthammer 2008).
Arching failure, shown in Figure 71, can be considered a kind of flexural failure, but it only develops in special circumstances where the wall foundation is perfectly rigid. The foundation must have sufficient strength and stiffness to resist rotation of the wall about the foundation connection. Arching is not likely to develop on a rock foundation with an unconfined compressive strength less than 5,000 psi. Few of the typical foundation rocks and none of the typical coals shown in Table 26 meet this minimum compressive strength consideration for arching to develop in a coal mine seal. As was argued earlier, the foundation rocks for most coal mine seals are fireclays, claystones, and shales. Development of the arching mechanism is not likely and therefore arching should be neglected.

![Figure 71. Arching of flexural members due to rigid external restraint.](image)

Internal arching can occur in deep beams where a substantial portion of the load is carried to the support through a compression strut (Rogowsky and MacGregor 1983). There are various definitions of deep beams. The American Concrete Institute (ACI) considers a beam deep if the span-to-thickness ratio is 5 or less (ACI 2008). ACI has provisions for deep beams to be analyzed using strut and tie methods as shown in Figure 72. The strut is a compression member within the concrete, usually the concrete itself, and the tie, labeled T in Figure 72, is a tension member, usually tensile reinforcing steel. The strut-and-tie method for analysis and design of deep concrete beams is a recognized design method that can apply to coal mine seal design (ACI 2008). The method is mentioned for completeness; however, this report uses other methods for analysis and design of deep beams.

### 6.1.2 Analysis of reinforced concrete seal design using the equivalent static method

This example illustrates the procedure to analyze and design a reinforced concrete beam using the equivalent static method and basic analysis equations presented in UFC 3-340-02 (DOD 2008). The approximate seal
geometry is shown in Figure 49 and is the same one considered earlier in Chapter 5 for the analysis and design of a hitch-type seal foundation. To reiterate, the seal material properties, the coal mine entry geometry, and the design pressure-time curve are as follows.

- Concrete compressive strength – $f_c' – 5,000$ psi
- Reinforcing steel yield strength – $f_y – 60,000$ psi
- Entry width – 16 ft
- Entry height – 4 ft
- Design pressure-time curve – 120 psi with instantaneous rise time
- Hitch depth (ribs and floor) – 3 ft

The equivalent static method, as discussed in Chapter 3, uses a dynamic load factor (DLF) to transform a dynamic structural analysis problem into an equivalent static analysis problem. The actual dynamic load will produce a certain maximum displacement in a structure. Multiplying the actual dynamic load by the DLF gives an equivalent static load, which will produce the same displacement in the structure. From Table 3, the DLF for the 120-psi design pressure-time curve with instantaneous rise time is 2.0.
When designing blast-resistant structures, it is a typical practice to increase the design charge weight by a factor of 20% to account for variations in the energy output of explosives. Alternatively, the design blast pressure can be increased by 20% to achieve the same effect. This 20% increase in charge weight or blast pressure can be viewed as a kind of safety factor (SF).

Design pressure \( P = 120 \cdot SF \cdot DLF \) = 120 \( \cdot 1.2 \cdot 2 = 288 \) psi

The dynamic strength of materials increases above the static strength depending on the strain rate of the applied dynamic load. From Tables 5 and 6, the dynamic increase factor (DIF) ranges from 1.00 to 1.19, depending on the kind of applied stress. This analysis assumes that the reinforcing steel acts in tension with a DIF of 1.00, and the concrete acts in compression with a DIF of 1.12. The design strengths are as follows.

Dynamic yield strength of steel \( f_{dy} = f_y \cdot 1.00 = 60,000 \cdot 1.00 = 60,000 \) psi

Dynamic yield strength of concrete \( f_{cc} = f'_c \cdot 1.12 = 5,000 \cdot 1.12 = 5,600 \) psi

For analysis purposes, the seal is assumed to act in flexure as a one-way wall from rib to rib across the width of the coal mine entry. Any restraint by anchorage or friction provided by the roof and floor is neglected. The entry is 16 ft wide, and the hitch depth is 3 ft on each rib. A conservative span (\( L \)) is

Span \( L = 16 + 3 + 3 = 22 \) ft

The distributed load on a 1-ft-wide strip of this 22-ft span is

\[ W_1 = 288 \text{ psi} \cdot 12 \text{ in.} = 3,456 \text{ lb/in.} = 41,472 \text{ lb/ft} \]

The maximum moment at the center of a span is calculated as

\[ M_m = \frac{W_1 \cdot L^2}{8} = \frac{41,472 \cdot 22^2}{8} = 2,509,056 \text{ lb-ft} \]

The ultimate resisting moment (\( M_u \)) for a rectangular section of width \( b \) with tensile reinforcement only is (DOD 2008, page 1076)
\[ M_u = \left( \frac{A_s f_{dy}}{b} \right) \left[ d - \left( \frac{a}{2} \right) \right] \]  
(47)

\[ a = \frac{A_s f_{dy}}{(0.85 \cdot b \cdot f'_{dc})} \]  
(48)

\[ p = \frac{A_s}{b \cdot d} \]  
(49)

where

- \( A_s \) = area of tension reinforcement within the width \( b \)
- \( f_{dy} \) = dynamic strength of steel reinforcement
- \( b \) = width of compression face
- \( d \) = distance from extreme compression fiber to centroid of tension reinforcement
- \( a \) = depth of equivalent rectangular stress block
- \( f'_{dc} \) = dynamic ultimate compressive strength of concrete
- \( p \) = reinforcement ratio, i.e., area of tension reinforcement over the concrete section area

Equations 47, 48, and 49 can be combined and re-arranged to solve for the area of steel reinforcement \( A_s \).

\[ A_s = \frac{M_u \cdot b}{f_{dy} \cdot d} \cdot \frac{1}{\left[ 1 - 0.59 \cdot p \cdot \frac{f_{dy}}{f'_{dc}} \right]} \]  
(50)

The ultimate resisting moment, \( M_u \), must exceed the maximum moment, \( M_m \), by some factor \( \phi \).

\[ M_u \geq \phi \cdot M_m \]  
(51)

Substituting Equation 51 into Equation 50 gives an equation for the required minimum area of reinforcement steel

\[ A_s' \geq \frac{\phi \cdot M_m \cdot b}{f_{dy} \cdot d} \cdot \frac{1}{\left[ 1 - 0.59 \cdot p \cdot \frac{f_{dy}}{f'_{dc}} \right]} \]  
(52)
Knowing the maximum moment, \( M_m \), and the material strengths, equation 52 can be solved iteratively until a suitable design is created and the beam depth \( d \) and amount of steel reinforcement given by \( A_s \) is specified.

As an initial trial design, assume that the seal thickness is 12 ft. Assuming that the reinforcing bar is covered with 2.5 in. of concrete, the distance \( d \) from extreme compression fiber to centroid of tension reinforcement is

\[
d = T - 2.5 \text{ in.} = 144 - 2.5 = 141.5 \text{ in.}
\]

Assume steel reinforcement using \#10 bars on 4-in. center-to-center spacing. For a 12-in.-wide width of compression face \( b \), the actual area of steel is

\[
A_s = 3 \cdot \frac{\pi}{4} \cdot \text{diameter}^2 = 3 \cdot \frac{\pi}{4} \left( \frac{10}{8} \right)^2 = 3.68 \text{ in.}^2
\]

The reinforcement ratio, \( p \), for the trial section is

\[
p = \frac{A_s}{b \cdot d} = \frac{3.68}{12 \cdot 141.5} = 0.00217
\]

The required minimum area of reinforcement steel from equation 52 is

\[
A_s' \geq \frac{\phi \cdot M_m \cdot b}{f_{dy} \cdot d} \cdot \frac{1}{1 - 0.59 \cdot p \cdot \frac{f_{dy}}{f_{dc}}}
\]

\[
= \frac{100 \cdot 2,509,056 \cdot 12}{60,000 \cdot 141.5} \cdot \frac{1}{1 - 0.59 \cdot 0.00217 \cdot \frac{60,000}{5,600}} = 3.60 \text{ in.}^2
\]

The actual area of steel, \( A_s = 3.68 \text{ in.}^2 \), is slightly larger than the required minimum area of reinforcement steel, \( A_s' = 3.60 \text{ in.}^2 \).

The final seal design is 12-ft thick with No. 10 reinforcing bars placed horizontally from rib-to-rib with 4-in. center-to-center spacing.
6.1.3 Analyses of reinforced concrete seals using WAC

These examples reconsider the reinforced concrete seal design derived previously using the equivalent static method of analysis with the single-degree-of-freedom (SDOF) method WAC. This method considers the inertia of the seal mass and results in a somewhat thinner and more economical design. The seal dimensions and seal properties are the same as summarized above. The seal design uses No. 10 reinforcing bars on 4-in. centers as before. This WAC analysis treats the seal as a one-way wall that is simply supported along the ribs.

For a 12-ft-thick seal, Figure 73 shows the calculated displacement versus time response to the 120-psi pressure-time curve with instantaneous rise time. From this response plot, the maximum displacement of the seal is 0.02744 in. The WAC analysis also calculates the elastic limit for the seal design as 0.0343 in. Because the maximum displacement is much less than the elastic limit, the seal thickness can be reduced.

Figure 74 shows the calculated displacement versus time response for a 9-ft, 10-in.-thick seal. The maximum displacement of the seal is 0.04805 in., and the elastic limit is 0.04886 in. Use of the SDOF method of analysis, which considers the inertia of the structure, has reduced the required seal thickness by 26 in. in this example.

The above analysis and design examples with WAC treated the seal as a one-way wall that is simply supported along the ribs. As a further example, the seal is analyzed as a two-way wall that is simply supported along the ribs and the floor. The seal is free to displace along the roof. Figure 75 shows the calculated displacement-versus-time response for this seal, which has now been reduced to 3-ft thick. This seal design contains the same steel reinforcement as before, i.e., No. 10 bars placed horizontally from rib-to-rib on 4-in. centers. The maximum displacement of the seal is 0.323 in., and the elastic limit is 0.340 in.

6.1.4 Validation of SDOF method (WAC) for reinforced concrete wall design and comparison of results of WAC analysis to explosion tests

SDOF analysis using WAC for reinforced concrete walls subjected to a blast load can provide very accurate results. To validate this analysis method, a results of a WAC model of a reinforced concrete wall is compared to experimental data from an actual blast test.
Figure 73. Calculated displacement-versus-time response for a 12-ft-thick seal analyzed as a one-way wall. Note that the maximum displacement is much less than the elastic limit.

As shown in Figures 76 and 77, each panel measures 64 in. by 33.75 in. by 4 in. thick. Figure 76 is a front view of the panel construction details, and Figure 77 is a side view of the reinforced concrete panels. The principal longitudinal reinforcing steel is on 4-in. centers, and the transverse steel is on 12-in. centers. The concrete cover on the front and back face is 1/2 in., and the edge cover is 3/4 in. To limit concrete damage at the panel edges, additional longitudinal reinforcing steel is added at 2-in. spacings near the two edges, as shown in Figure 76. The test panels are simply supported, one-way walls with a clear span of 52 in. in the test frame (Robert et al. 2011).
The panels were tested in the USACE Engineer Research and Development Center (ERDC) Blast Load Simulator facility. Figure 78 shows the measured pressure-time history on the wall. Peak pressure from the test blast is 51.3 psi, and the pressure has a duration of about 80 msec. Figure 78 also shows the impulse-time history from the experiment that is found by integrating the measured pressure-time history.

Figure 79 shows the measured acceleration-time history from the explosion test obtained from an accelerometer mounted at the centroid on the rear face of the test panel. Integrating the measured acceleration-time history twice gives the displacement-time history of the test panel, which is also shown on Figure 79. The measured peak displacement from the test was about 6.7 in. Figure 80 shows the test panel and the resulting damage after the explosion test. Visual inspection measured a final displacement of about 7 in., which agrees well with the calculated displacement from the accelerometer measurements.
Figure 75. Calculated displacement-versus-time response for a 3-ft-thick seal analyzed as a two-way wall simply supported along the ribs and floor. Note that the maximum displacement is slightly less than the elastic limit.

Figure 76. Front view of a reinforced concrete panel (from Robert et al. 2011).
The test panel was analyzed with WAC using the test panel dimensions and reinforcing steel details shown in Figures 76 and 77 along with the measured pressure-time history shown in Figure 78.

The calculated displacement-time history from WAC and the measured displacement-time history from the experiment are shown in Figure 81. There is excellent agreement between the experimental result and the SDOF calculation, especially in the early time. This experiment induced a displacement in the test panel that is far beyond its elastic limit. Support rotations were about 14 deg, which is well past complete failure of the test panel. Even at failure, the SDOF calculation with WAC is within about 20% of the observed maximum displacement. This validation demonstrates that excellent results can be obtained with WAC that is based on the simplified SDOF method of analysis.
6.2 Analysis of unreinforced concrete plug seals

In contrast to reinforced concrete seals that are relatively thin walls with a thickness-to-height ratio of about 1/2 or less, unreinforced concrete plug seals are relatively thick structures with a thickness-to-height ratio of about 1. Whereas reinforced concrete seals act primarily in flexure and fail by tensile failure of the reinforcement steel or compressive failure of the concrete, unreinforced concrete plug seals fail by other, more complex mechanisms. Even though unreinforced concrete plug seals are relatively thick, some flexure does develop that induces tensile and compressive forces in the structure. This analysis examines the failure mechanisms of unreinforced concrete plug seals.
Simple analytic methods for analysis of plug-type seals only consider a shear failure mechanism around the seal perimeter. In this analysis, ABAQUS (SIMULIA 2005) was used to examine the non-uniform distribution of tensile, compressive, and shear stresses within a relatively thick plug-type seal. Established design methods in the UFC 3-340-02 (DOD 2008) cannot be used directly to design an unreinforced concrete plug seal. These numerical analyses examine the limits and applicability of the traditional analytic methods for plug seal design.

For this ABAQUS analysis, the same model that was used in Chapter 5 to analyze the forces in rock-bolt anchors is used here to analyze the complex stress distributions within a thick, plug-type seal. The basic ABAQUS model of the unreinforced concrete plug seal with the applied load along...
with the rock-bolt anchors was shown in Figures 60 and 61. This plug seal is 84 in. thick and 84 in. high. The applied uniform pressure on the left side of the seal is 288 psi, which is the equivalent static pressure from the 120-psi design pressure-time curve with an instantaneous rise time, with a dynamic load factor of 2.0, and with a safety factor of 1.2. The model contains 392 quadratic, triangular, plane-strain solid elements (ABAQUS CPE6M), 112 linear line elements for the 14 rock-bolt anchors (ABAQUS B21), and a total of 996 nodes. Prior studies with coarser meshes showed this level of discretization gives accurate results.

The 84-in.-by-84-in. unreinforced concrete plug seal model shown in Figure 60 is divided into two pieces, i.e., an upper half and a lower half, separated by a frictionless contact surface in the middle of the model. This surface is completely artificial; however, it assures that the concrete has zero tensile strength and can only resist compressive forces. The concrete is modeled as linear elastic with a Young’s Modulus of 4,030,510 psi and a Poisson’s Ratio of 0.20.
Figure 81. Measured displacement-time history from an explosion test of a reinforced concrete panel compared to a computed displacement-time history using WAC.

Figure 82 shows stress in the y direction within the unreinforced concrete plug seal. The maximum compression stress in the y direction is about 394 psi along the extreme compression fiber that is parallel to the loaded left face. This stress is well below the 5,000-psi yield strength of the concrete. Along the artificial crack from the middle of the seal to the unloaded right face, the stress in the y direction is about zero. This artificial crack can transmit compressive (negative) stresses as it does on the left side of the structure, but it cannot transmit tensile (positive) stresses, hence the stresses in the y direction on the right side of the structure are near zero.
The Von Mises stress contours shown in Figure 83 clearly demonstrate the effect of arching within the concrete plug. Pressure applied to the left face of the plug is successfully transferred to the rock-bolt anchors around the perimeter of the seal through compression struts. Overall compression within the unreinforced concrete plug seal can be seen clearly in the plots of the minimum and maximum principal stress. The minimum principal stresses shown in Figure 84 are almost all compressive (negative) throughout the entire structure, except near the rock-bolt anchors near the loaded face where some tensile (positive) stresses appear. The maximum principal stresses shown in Figure 85 are also almost all compressive, except near the rock-bolt anchors.

The artificial crack across the centerline of the model was placed there to ensure that the unreinforced concrete cannot provide any tensile resistance. This modeling technique can be viewed as hypothetical but is necessarily conservative since it assures that any tensile strength of the unreinforced concrete is not considered in a design. The effect of this artificial crack is also seen in the plots of minimum and maximum principle stress. In Figure 84, the minimum principal stresses are all compressive along this artificial crack, and similarly in Figure 85, the maximum principal stresses are also always compressive along the centerline.
Figure 83. Von Mises stress contours for an unreinforced concrete plug seal.

Figure 84. Minimum principal stress contours for an unreinforced concrete plug seal.
Figure 85. Maximum principal stress contours for an unreinforced concrete plug seal.

The load travelling through the compression strut can be broken into the two Cartesian components. The foundation that supports the compression strut must be able to withstand the two components acting simultaneously. Since the load travels through the concrete structure at approximately a 45-deg angle with respect to the tunnel perimeter, the rock-bolt anchors will be subjected to combined stresses from the shear load at the interface and an axial load from the concrete plug.

Due to the extreme thicknesses of the proposed concrete plugs, diagonal and direct shear failures of the concrete are not an issue for any of the seal dimensions considered. The bending failure mechanism of the plug is very similar to a standard reinforced concrete beam, where the concrete resists crushing on the compression face, and the tension steel resists cracking on the tension face. However, the plug design benefits from the well known concept of arching, which utilizes external confinement rather than internal steel reinforcement to control tension cracking. In the concrete, a bending failure caused by a positive moment would create rotation of each half of the bifurcated concrete structure. The extreme tension fiber of this member is prevented from splitting away from the worst-case centerline crack by the restraining effects of the rock-bolt foundation. The extreme compression fiber is kept from inward rotation by the symmetrical contact between the top and bottom of the plug. As long as the concrete does not
crush along the compression face and the rock bolts prevent the outward movement of the tension face, the plug will remain in place.

For the mine seal to provide adequate protection, both the concrete plug and the rock-bolt foundation must respond elastically to the load. Since the concrete and steel materials were modeled as purely linear elastic, a check of the maximum stresses against the yield stress of each material will determine the adequacy of the design.

The results of the ABAQUS analysis show that the unreinforced concrete plug seal effectively resists the loading defined by the 120-psi pressure-time curve with an instantaneous rise time. The load is resisted by the shear and compressive strength of the concrete as well as the combined loading resistance of the rock-bolt supports. Due to the geometry and support conditions, the concrete plug even survives with a centerline crack that provides no shear or tensile resistance across the interface. This analysis shows that thick, unreinforced concrete plug seals, anchored by rock bolts to the surrounding foundation rock, can meet the requirements of the Final Rule (2008).

6.3 Analysis of gob plug seals

“Gob” is a general term for any waste rock in an underground coal mine. It is usually composed of broken pieces of shale, siltstone, or sandstone from the nearby coal mine roof rock. Seals constructed of gob from coal mining operations are a potentially inexpensive, low-logistics option for the coal mining industry to consider. A seal constructed from gob may be ready for service as soon as the seal is completed, since no time is required for the material to “cure” to reach its design strength. If subject to convergence between the mine roof and floor, a gob plug seal will exhibit increased shear resistance due to the increased confinement.

Figure 86 shows a conceptual gob plug seal that is constructed by piling gob in a coal mine entry and filling the entry completely over a specified length. Resistance of the gob plug seal to explosion forces is developed through frictional interlocking of the gob material with the surfaces of the surrounding roof, floor, and ribs. The ends of the gob plug seals may be left at the natural angle of repose of the material, or they could be contained between two walls called “load collectors”, as shown in Figure 86. These walls may be constructed from light materials such as steel mesh or reinforced cloth, or they may be actual walls constructed from concrete or
concrete blocks. The end wall option has the advantage of providing a uniform surface to load the gob plug seal and confining it—essentially loading the gob like a piston. Depending on its particle size distribution, the gob might dilate (expand) under load, thereby increasing the frictional resistance along the entry surfaces at the roof, floor, and ribs. Unlike traditional seals, a gob plug seal may compact under explosion loading, resulting in displacement of the loaded face but with little to no movement of the opposite face.

Figure 86. Schematic of a gob plug seal concept with 1-ft-thick load collector walls on either side to contain the gob.

Gob plug seals respond by absorbing the initial, instantaneously applied explosion shock load, and then resisting the subsequent, 4-sec-long, explosion gas pressure load statically. The initial part of the loading is analogous to airblast-induced ground shock, where the airblast overpressure from an explosion is transferred into the ground as a compression stress wave. This phenomenon has been studied extensively by the military for protective construction applications and for determining the vulnerability of structures to weapon detonations. Design manuals such as UFC 3-340-2 (DOD 2008), TM 5-1300 (Departments of the Army, the Navy, and the Air Force 1990), and TM 5-855-1 (Department of the Army et al. 1998) provide information on ground shock phenomena and attenuation rates in various geologic materials. The ground shock magnitude is attenuated by the crushing of the void volume within the soil, or any particulate matrix, by the passage of the stress wave. Materials of low density and high volumes of air-filled voids will attenuate ground shock better than materials of high density and low air-void volume (Priddy and Hodo 2009). Crushing of the
void volume by the ground shock requires that the voids be air-filled. The material must drain well, since water-filled voids would not be compressible, and therefore little or no attenuation would occur. Typical coal mine gob can fit these criteria well.

The engineering design question for gob plug seals is what length of seal is required to resist the design loads specified in the Final Rule (2008). Guidance for design of gob plug seals is minimal, and little data exist for analysis of this type of seal. Lusk et al. (2009) studied a gob plug seal design where the seal was constructed by blasting the roof rock into the entry. Sapko et al. (2010) studied gob piles as a method to reduce blast loads on seals. The large-scale tests in these studies provided some information on the behavior of gob plug seals as structures subjected to blast-wave loading. In order to estimate the length of gob piles that may effectively resist the MSHA design loads, a finite element analysis was conducted using gob material properties developed from experiments by Pappas and Mark (1993) and coal mine rock properties from Zipf (2006).

Pappas and Mark (1993) performed laboratory-scale, uniaxial compression tests on three kinds of simulated gob material, i.e., shale, weak sandstone, and strong sandstone, to develop material properties for modeling longwall coal mines. Their experiments determined the secant and tangent modulus values of the stress-strain curve for gob material. These tests provide an opportunity to compare the material models used in the finite element analysis for this study with the uniaxial compression test data. This comparison provides a level of confidence in the analysis procedure.

To create the simulated gob material for the laboratory-scale tests, an analysis of photographs of actual gob from three different mines was used to determine the size distribution of full-scale gob. Figure 87 shows the average gradation curve for the actual gob, which has an average particle size of more than 300 mm. This gradation curve was reduced by shifting it parallel to itself and downward to an average particle size of about 45 mm, which is the “shifted” curve in Figure 87. This shifted curve was then corrected to account for hidden particles in the photo analysis, leading to the “corrected” curve in Figure 87. The corrected gradation curve became the target for the laboratory-scale simulated gob used in the uniaxial compression tests. The material has a void ratio of about 30%, and 80% of the material has a size greater than 20 mm.
The full-scale shale gob material, obtained from a roof fall at a mine in Pennsylvania, has a compressive strength between 5,400 psi and 10,500 psi and an average density of 162 lb/cu ft. The material model for the shale gob material was modified from the weathered shale material model described earlier in Chapter 3 Table 15 so that it matched the laboratory-scale, uniaxial test results of the simulated gob. Figure 88 shows the average, stress-strain data for shale gob material compared to a finite element model of the laboratory test. There is good match between the calculation and the test results that provides confidence that the adjusted weathered shale model is representative of this material.

A finite element model for an 18-ft-thick gob plug seal was constructed that mimics the schematic shown in Figure 86. The model is axisymmetric with a circular cross-sectional area equivalent to the area of an entry 7-ft high by 20-ft wide. The surrounding foundation material is either weathered shale or competent shale. The calculation was performed dynamically using the 120-psi pressure-time curve with an instantaneous rise time and a 4-sec duration. Figure 89 shows the pressure in the gob plug seal at 0.75 sec for
the weathered shale case. The pressure is relatively uniform along the length of the plug with small pressure concentrations in the walls on both the loaded (left) and unloaded (right) ends of the model.

**Figure 88.** Calculated uniaxial stress-strain curve (green line) from finite element simulation of gob behavior compared to measured uniaxial stress-strain behavior (red squares) by Pappas and Mark (1993).

**Figure 89.** Pressure plot of the gob plug seal in a weathered shale foundation, showing an increased pressure at the loaded surface (left) and on the non-loaded wall (right).

Figure 90 shows the effective plastic strain in the plug at 0.75 sec. There is significant strain at the interface between the gob plug seal and the surrounding weathered shale foundation, indicating a slip of the seal. Figure 91 shows the displacement time-history on the loaded (left) side of
the plug compared to the unloaded (right) side of the plug. Both sides have undergone significant displacements at 0.75 sec, i.e., 24 in. on the loaded side and 14 in. on the unloaded side. This result indicates that the 18-ft-thick gob plug seal surrounded by weathered shale will fail because it is not thick enough or massive enough.

Figure 90. Effective plastic strain in the gob plug seal surrounded by weathered shale foundation, showing an increased effective plastic strain at the interface on the loaded end (left) compared to the non-loaded end (right).

Figure 91. Calculated displacement-time histories of the inby (loaded) end and outby end of a gob plug seal with a weathered shale foundation.
Figure 92 shows the pressure and Figure 93 shows the effective plastic strain in an 18-ft-thick gob plug seal surrounded by a competent shale foundation material. Figure 92 indicates greater stress in the surrounding competent shale foundation compared to the weathered shale foundation (Figure 89). Figure 93 indicates more plastic strain within the gob plug seal and much less plastic strain in the surrounding competent shale foundation, compared to the weathered shale foundation (Figure 90). Figure 94 shows the displacement time-history on the loaded (left) side of the plug compared to the unloaded (right) side of the plug with a competent shale foundation. In this case, the loaded surface (inby) displaced about 8.5 in., and the unloaded surface (outby) moved less than an inch.
More importantly, both faces reached a constant displacement at about 0.25 msec and came to rest. This result indicates that the 18-ft-thick gob plug seal surrounded by competent shale survives the blast loading. The only difference between the two calculations is the strength of the surrounding foundation materials.

In order to look at a more realistic behavior of a gob plug seal in an entry, a three-dimensional simulation was also conducted using LS-DYNA (LTSC 2012). The same material model was used for both the gob and the surrounding materials as in the previous axisymmetric calculations. The surrounding foundation material in this simulation is competent shale. Half-symmetry was used in constructing this model of the 18-ft-thick gob plug seal. As shown in Figure 95, a vertical symmetry plane exists along the thickness of the seal. The model seal is still 7 ft high but only 10 ft wide because of symmetry.

The results are similar to the previous axisymmetric calculation in that significant displacement of the loaded surface is observed, but the non-loaded surface does not exhibit any deflection, as shown in Figure 96. The displacement-time history of the loaded face, shown in Figure 97, indicates a permanent displacement of 5 in.
Figure 95. Finite element model of an 18-ft-thick gob plug seal in a 20-ft-wide-by-7-ft-high entry. The model uses 1/2 symmetry along a vertical plane through the seal.

Figure 96. Displacement in the gob plug seal at 400 msec after the initial loading.
The analyses presented here show that gob plug seals are a viable option for coal mine seals; however, not enough information exists to provide guidance for their design. In these simulations, containment walls constructed with concrete on each side of the seal were not attached to the surrounding rock foundation and did not provide significant resistance to the blast loading. The purpose of the walls is to contain the gob during construction and to provide an even loading of the gob like a piston. The walls also provide a gas seal. Containment walls made from gob materials could be used as long as they can survive repeated loading. To better model gob plug seals and develop design criteria, gob material properties must be better understood. Specifically, estimates of shear strength of gob material are needed. Barton (2008) provides equations for estimating the shear strength of rock fill used in dam construction and points out the importance of the interface conditions between the rock fill material (similar to gob) and a dam rock foundation (similar to the seal foundation walls). An important conclusion is that the shear strength at the interface between the rock fill and the foundation is non-linear and stress-dependent and that sufficient roughness at the interface is necessary to maximize shear strength. In addition to better understanding the shear strength properties of gob under a load, full-scale testing using realistic static and dynamic loads is needed to validate any potential gob plug seal designs.
7 Guidelines for Design of Coal Mine Seals

7.1 General procedure for coal mine seal design

Figure 98 shows a simple, general flowchart for coal mine seal design that applies to all seals, whether they are reinforced concrete walls, unreinforced concrete plug seals, or some other kind of seal.

The following sections describe detailed design procedures for reinforced concrete seals and unreinforced concrete plug seals. The procedures are illustrated with numerical examples, and design charts are presented for a wide range of conditions. Most of the calculations that are shown are performed within the Wall Analysis Code (WAC) and should be transparent to the designer. Those parts of the design procedure that require specific input or a decision by the designer are highlighted.
7.2 Design of reinforced concrete seals

Reinforced concrete seal design follows the general outline shown in Figure 98 with the following detailed steps.

- **A. Design Inputs**
  1. Specify design load
  2. Specify allowable failure
  3. Specify safety factor
  4. Determine dynamic load factor (DLF)
  5. Specify coal mine geometry
  6. Determine support conditions
  7. Specify material properties
  8. Determine dynamic increase factor (DIF)

- **B. Foundation Design (seal anchorage)**
  1. Determine approximate yield line location
  2. Determine approximate shear forces
  3. Determine number of rock-bolt anchors
  4. Determine minimum seal thickness based on foundation design

- **C. Seal Structure Design**
  1. Estimate reinforcing steel requirements
  2. Determine diagonal shear reinforcement
  3. Determine actual vertical moment capacity
  4. Determine actual horizontal moment capacity
  5. Determine static properties of the design
  6. Determine actual yield line location and ultimate resistance
  7. Determine direct shear capacity of concrete
  8. Determine dynamic response of seal

These detailed steps are illustrated in the following flowchart for reinforced concrete seal design. This flowchart closely follows protective structure design procedures presented in the design manual UFC 3-340-02 (DOD 2008).
7.2.1 Procedure for the design of reinforced concrete seals

7.2.1.1 Design inputs

Specify design load. The design load is found by choosing one of the design pressure-time curves specified in the Final Rule (2008) shown in Figures 1 through 4 and 2 of this report.

Specify allowable failure. Section 4-16 of UFC 3-340-02 (DOD 2008) discusses three types of reinforced concrete cross sections that can be utilized in the design and analysis of blast resistant concrete slabs, depending on the magnitudes of the blast output and allowable degree of damage to the slab and its residual load-bearing capacity.

1. Type I - The concrete is effective in resisting moment. The concrete cover over the reinforcement on both surfaces of the element remains intact.
2. Type II - The concrete is crushed and not effective in resisting moment. Compression reinforcement equal to the tension reinforcement is required to resist moment. The concrete cover over the reinforcement on both surfaces of the element remains intact.
3. Type III - The concrete cover over the reinforcement on both surfaces of the element is completely disengaged. Equal tension and compression reinforcement that is properly tied together is required to resist moment.

Because of the “elasticity of design” requirement for seals discussed in Chapter 2 of this report, seal design is limited to Type I only, where the seal remains intact and its response is limited to elastic deflection only. Therefore, maximum deflection equal to the elastic deflection limit controls seal design.

\[ X_{\text{max}} \leq X_{\text{elastic}} \quad (53) \]

Specify safety factor. In protective structure design, the design load is typically specified indirectly through an equivalent TNT charge weight, which, in turn, implies some design pressure-time curve. Two separate but related factors are applied that affect the design load for the structure. First, the charge weight increase factor accounts for uncertainty in the maximum explosion pressure. Second, the dynamic load factor accounts for the increase in the applied load due to the rapidity with which the load is applied.
Section 1-7 of UFC 3-340-02 states that the equivalent TNT charge weight should be multiplied by \( \phi_p = 1.2 \) as a safety factor. Conservative assumptions and methods throughout the rest of the design process provide additional safety. Assuming that the charge weight has a linear effect on peak pressure, the 1.2 safety factor can be applied directly to the peak pressure in the design pressure-time curves in the Final Rule (2008).

\[
P_d' = P_d \cdot \phi_p
\]  

(54)

where \( P_d \) is the peak dynamic pressure and \( P_d' \) is the adjusted dynamic pressure.

Determine dynamic load factor (DLF). When utilizing the Equivalent Static Method of Analysis discussed in Section 3.2 of this report, the DLF for transforming the applied dynamic load into an equivalent static load depends on the rise time of the design pressure-time curve. The DLF for the pressure-time curves in the Final Rule (2008) is either 2.0 or 1.0, as presented in Tables 3 and 4 of this report. The equivalent static design pressure is

\[
P_s = P_d' \cdot DLF
\]  

(55)

Specify the coal mine entry geometry. The width, \( W \), for a typical coal mine entry is 20 ft, and it may range from about 16 to 24 ft in common practice. The height, \( H \), of a coal mine entry may range from about 3 to 17 ft, and is typically about 7 ft.

Determine the support conditions. As discussed in Chapter 5 of this report, the foundation rocks surrounding a seal are normally weak and unlikely to provide any rotational (moment) resistance to displacement of the seal structure. A simple support condition applies to all four sides of the seal (roof, floor, and ribs). This support condition means that the sides can resist movement or translation but not bending or moment. This simple support condition on all four sides is designated (SSSS).

Specify material properties. Reinforced concrete seal design requires the compressive strength of concrete (\( f'_c \)) and the yield strength of rebar and rock bolts (\( f_y \)). Other important design specifications include the clear cover in wall (\( CC_w \)), the stirrup diameter (\( \Phi_{sw} \)), the vertical rebar diameter
(Φ_vw) and its spacing (s_v), and the horizontal rebar diameter (Φ_hw) and its spacing (s_h).

**Determine the dynamic increase factor (DIF)**. As discussed in Section 3.2 of this report, construction materials subject to dynamic loading typically exhibit higher strength than materials subject to ordinary static loading. Tables 5 through 9 of this report, extracted from the UFC 3-340-02, summarize most of the DIF values used in this report.

### 7.2.1.2 Foundation design (seal anchorage)

**Determine the approximate yield line location**. Since the resistance, r_u, is greater than or equal to the load, P_s, the support reactions at the wall are determined by the load and not the resistance. The location x of the yield line can be approximated by assuming that the vertical and horizontal moment capacities (M_{vp} and M_{hp}) are equal. This assumption is acceptable if the design specifies the same amount of reinforcement in both the vertical and horizontal directions.

For two-way walls with four edges supported, UFC 3-340-02 provides relations for the ultimate resistance of the wall in terms of its geometry and moment capacity (UFC 3-340-02.)

\[
\begin{align*}
r_u &= \frac{(M_{vp} + M_{vn})(6L - 2x)}{y^2(3L - 4x)} \\
r_u &= \frac{(M_{vp} + M_{vn})(6L - 2x)}{(H - y)^2(3L - 4x)} \\
r_u &= \frac{SM_{hp}}{x^2}
\end{align*}
\]

where:

- \(M_{vn}\) = vertical positive moment capacity
- \(M_{hp}\) = horizontal positive moment capacity
- \(M_{vn}\) = vertical negative moment capacity
- \(L\) = width of seal
- \(H\) = height of seal
- \(x\) and \(y\) = horizontal and vertical yield line locations.
Equations 56, 57, and 58 can be solved for the horizontal location of the yield line \( x^* \). Because of symmetry, the vertical location of the yield line is at mid-height, \( y = H/2 \). Therefore, the same result is obtained when substituting \( y = H/2 \) in equations 56 and 57. Because the support conditions at all edges are simple, the vertical and horizontal negative moment capacities are zero. Substituting for \( y \), equating equations 57 and 58, and recalling the assumption that \( M_{vp} = M_{hp} \), a cubic expression results for the location of the yield line.

\[
0 = 2x^3 - 6Lx^2 - 5H^2x + \frac{15}{4}H^2L
\]  
(59)

The horizontal location of the yield line (\( x \)) can be found by solving for the unique real root of equation 59. The \( x^* \) symbol is used to signify an approximate value for \( x \) calculated with the assumption of equal horizontal and vertical moment capacities, as opposed to an actual value for \( x \) calculated by using the actual moment capacities without this assumption.

**Determine the approximate shear forces.** Having determined an approximate value for the horizontal location of the yield line, \( x^* \), the shear forces along the roof, floor, and walls of the coal mine seal are calculated. (Recall that this assumption is acceptable as long as the amount of reinforcement in both directions is approximately equal.) For two-way walls with four edges supported, UFC 3-340-02 provides relations for the ultimate shearing support of the wall in terms of its geometry and ultimate resistance, \( r_u \). (See UFC 3-340-02.)

The approximate shear forces per foot of wall at the roof, floor, and ribs are

Shear at roof \( V_{sv, r}^* = \frac{3P_s(y(2L - 2x^*) \cdot 12}{(6L - 2x^*)} \)  
(60)

Shear at floor \( V_{sv, f}^* = \frac{3P_s(H - y)(2L - 2x^*) \cdot 12}{(6L - 2x^*)} \)  
(61)

Shear at ribs \( V_{sv, h}^* = \frac{3P_s x^* \cdot 12}{5} \)  
(62)
For seal foundation design, the maximum shear force per foot of wall is used.

\[
V_{u}^{*} = \max \left\{ V_{sV}, V_{sVf}, V_{sH}^{*} \right\}
\]  
(63)

Determine the number of rock-bolt anchors. For practical reasons, the minimum spacing of rock-bolt anchors, \( S_{RB} \), is assumed to be 12 in. in both directions. The shear strength of steel rock bolts is taken to be 60% of the yield strength of the steel, as discussed in UFC 3-340-02 and presented in Table 6 of this report. Also see ACI 318-08, Section 11.6, Equation 11-25 for the shear strength calculation.

The shear strength of a single rock-bolt anchor is

\[
F_{nv} = \mu_{RB} f_{yRB}
\]  
(64)

where \( f_{yRB} \) is the yield strength of the rock-bolt anchor steel and \( \mu_{RB} \) is the shear strength reduction factor, taken to be 0.6.

The area of a rock-bolt anchor with diameter \( \phi_{d} \) is

\[
A_{\phi_{d}} = \frac{\pi \phi_{d}^{2}}{4}
\]  
(65)

The approximate number of rock-bolt anchors required of diameter \( \phi_{d} \) per foot of wall is

\[
N_{\phi_{d}}^{*} = \frac{V_{u}^{*}}{F_{nv} A_{\phi_{d}}}
\]  
(66)

Note – Round up to the next integer value.

Determine minimum seal thickness. The minimum seal thickness, \( T_{c}^{*} \), must provide adequate room for 12-in. center-to-center rock-bolt anchor spacing plus 1.5-in. clear cover on both sides.

\[
T_{c}^{*} = \left( N_{\phi_{d}}^{*} - 1 \right) \cdot 12 + \phi_{d} + 3 \text{ in.}
\]  
(67)
7.2.1.3 Seal structure design

Estimate reinforcing steel requirements. Based on Equation 58 above, the required horizontal and vertical positive moment capacities are approximately

\[ M'_{HP} = M'_{VP} = \frac{P_s (x^*)^2}{5} \quad (68) \]

where \( P_s \) is the equivalent static pressure from Equation 55 and \( x^* \) is the horizontal location of the yield line.

The approximate distances from the extreme compression fiber to the centroid of tension reinforcement in the vertical and horizontal directions are

\[ d'_v = T'_c - 2.5 \text{ in.} \quad (69) \]
\[ d'_H = T'_c - 3.5 \text{ in.} \quad (70) \]

The ultimate resisting moment for a rectangular section of width \( b = 12 \text{ in.} \) with tensile reinforcement only, is given by the following equations (UFC 3-340-02, or Chapter 6 this report, Equations 47 to 49). The vertical positive moment capacity is

\[ M^*_{VP} = \left( A_{Vs}^* \frac{f_{dy}}{b} \right) \left( d'_v - \frac{A_{Vs}^* f_{dy}}{0.35(12) f'_{dc}} \right) \quad (71) \]

The horizontal positive moment capacity is

\[ M^*_{HP} = \left( A_{Hs}^* \frac{f_{dy}}{b} \right) \left( d'_H - \frac{A_{Hs}^* f_{dy}}{0.35(12) f'_{dc}} \right) \quad (72) \]

These equations can be rearranged to solve for the area of vertical and horizontal reinforcement as follows.
\[ A^*_v = \frac{M^*_{vp}}{f_{dy} \cdot d'_v} \cdot \frac{1}{1 - 0.59 \cdot \rho_v \cdot \frac{f_{dy}}{f'_{dc}}} \tag{73} \]

\[ \rho_v = \frac{A^*_v}{b \cdot d'_v} \tag{74} \]

\[ A^*_h = \frac{M^*_{hp}}{f_{dy} \cdot d'_h} \cdot \frac{1}{1 - 0.59 \cdot \rho_h \cdot \frac{f_{dy}}{f'_{dc}}} \tag{75} \]

\[ \rho_h = \frac{A^*_h}{b \cdot d'_h} \tag{76} \]

where

- \( A^*_v \) = area of vertical tension reinforcement within the width \( b = 12 \) in.
- \( A^*_h \) = area of horizontal tension reinforcement within the width \( b = 12 \) in.
- \( \rho_v \) = reinforcement ratio (area of vertical tension reinforcement to area of concrete in compression)
- \( \rho_h \) = reinforcement ratio (area of horizontal tension reinforcement to area of concrete in compression)
- \( f_{dy} \) = dynamic strength of steel reinforcement
- \( b \) = width of compression face = 12 in.
- \( d'_v \) = distance from extreme compression fiber to centroid of vertical tension reinforcement
- \( d'_h \) = distance from extreme compression fiber to centroid of horizontal tension reinforcement
- \( f'_{dc} \) = dynamic ultimate compressive strength of concrete.

Equations 73 and 74 can be solved iteratively for \( A^*_v \), and Equations 75 and 76 can be solved iteratively for \( A^*_h \).

The approximate minimum diameter of the vertical and horizontal reinforcement steel is
Determine diagonal shear reinforcement. In addition to bending moment, a seal must also resist shear forces. According to accepted design manuals, the shear strength of unreinforced concrete may be calculated with either Equation 79 or 80. (UFC 3-340-02, or ACI 318-08, Section 11.11.3.1)

\[
v_c = 2(f'_{dc})^{0.5}
\]

(79)

\[
v_c = [1.9(f'_{dc})^{0.5} + 2500p] \leq 3.5(f'_{dc})^{0.5}
\]

(80)

where \( p \) is the reinforcement ratio defined by Equation 74 or 76.

The shear resistance of unreinforced concrete per foot of wall is

\[
V_c = 12 \cdot v_c \cdot T_c^*
\]

(81)

If the value of \( V_u^* \) found with Equation 63 is greater than \( V_c \), then the amount of shear load that must be resisted by shear reinforcement is

\[
V_S = V_u^* - V_c
\]

(82)

UFC 3-340-02 Section 4-18.4 states that for slabs “at least one stirrup must be located at each bar intersection.” Therefore, the minimum spacing (s) is 6 in. The required area of shear reinforcement with stirrups is

\[
A_{s_{min}} = \frac{(V_u^* - V_c)s_s}{d_v(f_{dy})}
\]

(83)

where \( s_s \) is the stirrup spacing.

The minimum diameter of the stirrups is
Specify stirrup diameter \( \phi_{sw} \geq \phi_{s\text{min}} \).

Determine actual vertical moment capacity. Specify vertical reinforcement diameter \( \phi_{Vw} \geq \phi_{V\text{min}} \).

The area of vertical tension reinforcement steel per foot of wall is

\[
A_{Vs} = \frac{\pi \cdot \phi_{Vw}^2}{4} \cdot \frac{12}{s_V}
\]

(85)

where \( s_V \) is the center-to-center spacing of the vertical tension reinforcement.

The actual distance from the extreme compression fiber to the centroid of the vertical tension reinforcement is

\[
d_V = T_c - \left( CC_w + \Phi_{sw} + \frac{\Phi_{Vw}}{2} \right)
\]

(86)

where

\[
T_c = \text{seal thickness} \approx Tc^* \text{ from Equation 67}
\]

\[
CC_w = \text{concrete cover over reinforcement (usually taken as 1.5 in.)}
\]

\[
\Phi_{sw} = \text{actual diameter of stirrups for diagonal shear reinforcement}
\]

\[
\Phi_{Vw} = \text{actual diameter of vertical tension reinforcement}
\]

Calculate the actual reinforcement ratio for vertical tension reinforcement steel.

\[
\rho_V = \frac{A_{Vs}}{b \cdot d_V}
\]

(87)

where \( b \) is the width of the compression face = 12 in.
To insure against sudden compression failures, the maximum reinforcement ratio \( \rho \) must not exceed 75% of the ratio \( \rho_b \), which produces balanced conditions at the ultimate strength of the steel (UFC 3-340-02).

\[
\rho_b = \left( 0.85 \cdot K_1 \cdot \frac{f'_{dc}}{f_{dy}} \right) \frac{87000}{(87000 + f_{dy})} \quad (88)
\]

\[
K_1 = 0.85 - 0.05 \left( \frac{f'_{dc} - 4000}{1000} \right) \quad (89)
\]

The minimum reinforcement ratio (UFC 3-340-02) is

\[
\rho_{s min V} = \frac{1.25 \sqrt{f'_{dc}}}{f_{dy}} \quad (90)
\]

The actual vertical reinforcement ratio, \( \rho_V \), must meet the following condition.

\[
\rho_{s min V} \leq \rho_V \leq \rho_b \quad (91)
\]

The depth of an equivalent rectangular compression block is

\[
a_V = \frac{A_{Vs} \cdot f_{dy}}{0.85 \cdot b \cdot f'_{dc}} \quad (92)
\]

The vertical ultimate moment capacity per linear foot of wall (UFC 3-340-02) is

\[
M_{Vu} = \left( A_{Vs} \cdot \frac{f_{dy}}{b} \right) \left( d_v - \frac{a_v}{2} \right) \quad (93)
\]

where \( b \) is the width of the compression face = 12 in.

Due to the simple support conditions, the negative moment capacity \( M_{Vn} \) is zero, and the positive moment capacity \( M_{Vp} \) equals the vertical ultimate moment capacity \( M_{Vu} \).
Determine actual horizontal moment capacity. Specify the horizontal reinforcement diameter $\Phi_{Hw} \geq \Phi_{Hmin}$.

The area of horizontal tension reinforcement steel per foot of wall is

$$A_{Hs} = \left( \frac{\pi \cdot \Phi_{Hw}^2}{4} \right) \cdot \frac{12}{s_H} \quad (94)$$

where $s_H$ is the center-to-center spacing of the horizontal tension reinforcement.

The actual distance from the extreme compression fiber to the centroid of the horizontal tension reinforcement is

$$d_H = T_c - \left( CC_w + \Phi_{sw} + \Phi_{vw} + \frac{\Phi_{Hw}}{2} \right) \quad (95)$$

where

- $T_c$ = seal thickness $\approx T_c^*$ from equation 67
- $CC_w$ = concrete cover over reinforcement (usually taken as 1.5 in.)
- $\Phi_{sw}$ = actual diameter of stirrups for diagonal shear reinforcement
- $\Phi_{vw}$ = actual diameter of vertical tension reinforcement
- $\Phi_{Hw}$ = actual diameter of horizontal tension reinforcement.

Calculate the actual reinforcement ratio for horizontal tension reinforcement steel.

$$\rho_H = \frac{A_{Hs}}{b \cdot d_H} \quad (96)$$

where $b$ is the width of the compression face $= 12$ in.

The minimum reinforcement ratio (UFC 3-340-02) is

$$\rho_{s,\text{min}.H} = \frac{1.25 \sqrt{f_d^* \cdot b \cdot d_H}}{f_{dy}} \quad (97)$$
The actual horizontal reinforcement ratio $\rho_H$ must meet the following condition.

$$\rho_{s,\text{min},H} \leq \rho_H \leq \rho_b$$ \hfill (98)

The depth of an equivalent rectangular compression block is

$$a_H = \frac{A_{Hs} \cdot f_{dy}}{0.85 \cdot b \cdot f'_{dc}}$$ \hfill (99)

The horizontal ultimate moment capacity per linear foot of wall (UFC 3-340-02) is

$$M_{Hu} = \left( A_{Hs} \cdot \frac{f_{dy}}{b} \right) \left( d_H - \frac{a_H}{2} \right)$$ \hfill (100)

where $b$ is the width of the compression face = 12 in.

Due to the simple support conditions, the negative moment capacity $M_{Hn}$ is zero, and the positive moment capacity $M_{Hp}$ equals the vertical ultimate moment capacity $M_{Hu}$.

Determine static properties of the design. Specify the modulus of elasticity ($E_s$) for steel as 29,000,000 psi and Poisson’s ratio for steel as 0.30.

The modulus of elasticity for concrete is

$$E_c = w_c^{1.5} \cdot 33 \cdot \left( f'_{c} \right)^{0.5}$$ \hfill (101)

The Poisson’s ratio for concrete is 0.167 (UFC 3-340-02).

The modulus ratio is

$$n = \frac{E_s}{E_c}$$ \hfill (102)

The average reinforcement ratio is
\[ \rho = \frac{\rho_v + \rho_H}{2} \] (103)

The moment of inertia for a slab with unit width is

\[ I_y = \frac{T_c^3}{12} \] (104)

Using the average reinforcement ratio \( \rho \) from Equation 103 and the modular ratio \( n \) from Equation 103, determine the cracked concrete coefficient (\( F \)) from Figure 99.

**Figure 99.** Cracked concrete coefficient (\( F \)) as a function of the reinforcement ratio \( \rho \) and the modular ratio \( n \) (UFC 3-340-02, Figure 4-12).

The cracked moment of inertia for a slab with unit width is

\[ I_c = F \left( \frac{d_v + d_H}{2} \right)^3 \] (105)

The averaged moment of inertia is
The flexural rigidity is

\[ D = \frac{E_c \cdot I_a}{1 - v^2} \]  

(107)

Determine the seal’s height-to-width ratio \( H/L \). Note that \( H < L \). Using the \( H/L \) ratio, determine the constants \( B_{1V}, \beta_{1H}, \) and \( \gamma_1 \) for the structure using Figure 100.

**Figure 100.** Moment and deflection coefficients for a two-way wall with all edges simply supported (UFC 3-340-02, Figure 3-36).
Determine the actual yield line location and the ultimate resistance. Using the actual vertical moment capacity $M_{vp}$ from Equation 93 and the actual horizontal moment capacity $M_{Hp}$ from Equation 100, determine the actual yield line location $x$ and the ultimate elastic resistance $r_u$ by resolving Equations 56 and 58 presented earlier.

$$r_u = \frac{M_{vp}(6L-2x)}{\left(\frac{H}{2}\right)(3L-4x)}$$  \hspace{1cm} (108)

$$r_u = \frac{5M_{Hp}}{x^2}$$  \hspace{1cm} (109)

The elastic deflection of the seal at its ultimate elastic resistance is

$$X_e = \frac{\gamma_1 \cdot r_u \cdot H^4}{D}$$  \hspace{1cm} (110)

where $\gamma_1$ is determined from Figure 100.

The elastic stiffness is determined as

$$K_e = \frac{r_u}{X_e}$$  \hspace{1cm} (111)

Determine the direct shear capacity of concrete. The direct shear capacity of the concrete near the supports must exceed the maximum shear force in the concrete.

The direct shear capacity of concrete is (UFC 3-340-02)

$$V_d = 0.16 f'_{dc} \cdot b \cdot d_H$$  \hspace{1cm} (112)

Using the actual yield line location found by solving Equations 109 and 109, the actual shear forces per foot of wall at the roof, floor and ribs are

$$\text{Shear at roof } V_{slyf} = \frac{3P_y(2L-2x)\cdot12}{(6L-2x)}$$  \hspace{1cm} (113)
Shear at floor \( V_{svr} = \frac{3P_s(H - y)(2L - 2x) \cdot 12}{(6L - 2x)} \) (114)

Shear at ribs \( V_{sh} = \frac{3P_s x \cdot 12}{5} \) (115)

The maximum shear force per foot of wall is

\[
\text{maximum shear } V_u = \max \{ V_{svf}, V_{svr}, V_{sh} \} \]

(116)

\[
V_d \geq V_u
\]

(117)

Determine the dynamic response of seal. The maximum dynamic displacement of the seal and its natural period of vibration are based on the seal geometry, its properties, and its ultimate resistance. The elastic load mass factor, \( K_{LM}_{\text{elastic}} \), is determined using Table 28 and the seal dimensions \( L \) and \( H \). For most coal mine seals, the four sides are simply supported.

The plastic load mass factor, \( K_{LM}_{\text{plastic}} \), is determined using Figure 101 and the seal dimensions. Again, the four sides of a coal mine seal are usually simply supported. The effective load mass factor, \( K_{LM} \) is

\[
K_{LM} = \left( \frac{K_{LM}_{\text{elastic}} + K_{LM}_{\text{plastic}}}{} \right) \]

(118)

The effective unit mass is

\[
m_e = \frac{\rho_c \cdot T_c \cdot K_{LM}}{g}
\]

(119)

where

- \( \rho_c \) = density of concrete
- \( T_c \) = seal thickness
- \( g \) = acceleration due to gravity (32.2 ft per sec²)
- \( K_{LM} \) = effective load mass factor.
Table 28. Elastic load mass factor, $K_{LM_{\text{elastic}}}$, for two-way elements (UFC 3-340-02, 2008, Table 3-13).

<table>
<thead>
<tr>
<th>Support Conditions</th>
<th>Value of $L/H$</th>
<th>Elastic and Elasto-Plastic Ranges (Support Conditions)</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>All Supports Fixed</td>
<td>One Support Simple Other Supports Fixed</td>
<td>Two Supports Simple Other Supports Fixed</td>
<td>Three Supports Simple Other Supports Fixed</td>
<td>All Supports Simple</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Two adjacent edges supported and two</td>
<td>$L/H &lt; 0.5$</td>
<td>0.65</td>
<td>0.66</td>
<td></td>
<td></td>
<td></td>
<td>0.66</td>
<td></td>
</tr>
<tr>
<td>edges free</td>
<td>$0.5 \leq L/H \leq 2$</td>
<td>0.65 $- 0.16 \frac{L}{2H} - 1$</td>
<td>0.66 $- 0.144 \frac{L}{2H} - 1$</td>
<td>0.65 $- 0.186 \frac{L}{2H} - 1$</td>
<td></td>
<td></td>
<td>0.79</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$L/H \geq 2$</td>
<td>0.65</td>
<td>0.66</td>
<td>0.65</td>
<td></td>
<td></td>
<td>0.79</td>
<td></td>
</tr>
<tr>
<td>Three edges supported and one edge</td>
<td>$L/H = 1$</td>
<td>0.61</td>
<td>0.61</td>
<td>0.62</td>
<td></td>
<td></td>
<td>0.79</td>
<td></td>
</tr>
<tr>
<td>free</td>
<td>$1 \leq L/H \leq 2$</td>
<td>0.61 $+ 0.16 \left( \frac{L}{H} - 1 \right)$</td>
<td>0.61 $+ 0.16 \left( \frac{L}{H} - 1 \right)$</td>
<td>0.62 $+ 0.16 \left( \frac{L}{H} - 1 \right)$</td>
<td>0.63</td>
<td>0.63</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$L/H \geq 2$</td>
<td>0.77</td>
<td>0.77</td>
<td>0.78</td>
<td></td>
<td></td>
<td>0.79</td>
<td></td>
</tr>
</tbody>
</table>
The natural period of vibration is

$$T_N = 2\pi \left( \frac{m_e}{K_e} \right)^{0.5}$$  \hspace{1cm} (120)

The maximum dynamic displacement of the seal is

$$x_m = x_e \cdot \frac{X_m}{X_e}$$  \hspace{1cm} (121)
where

\[ x_e = \text{the elastic deflection of the seal at ultimate elastic resistance (equation 112)} \]

\[ X_m/X_e = \text{a parameter determined from response chart.} \]

The parameter \( X_m/X_e \) is determined with the response chart shown in Figure 102. The parameters for this chart are \( r_u/P_d \) and \( T/T_N \) where \( r_u \) is the ultimate elastic resistance from Equations 108 and 109, \( P_d \) is the adjusted dynamic load from Equation 54, \( T \) is the load duration (taken as 4 sec), and \( T_N \) is the natural period of vibration from Equation 119.

**Figure 102. Maximum deflection of an elastic-plastic, single-degree-of-freedom system for a rectangular load (UFC 3-340-02, 2008, Figure 3-56).**
Because coal mine seal design is always in the elastic range, the elastic resistance ($r_e$) must be equal to or greater than twice the adjusted dynamic load ($P_d'$), due to the dynamic load factor (DLF) of 2. The maximum dynamic displacement will be less than or equal to the elastic deflection at ultimate elastic resistance.

### 7.2.2 Numerical design example for a reinforced concrete seal

The following numerical example illustrates the design procedure for reinforced concrete seals presented in Section 7.2.1 of this report.

#### 7.2.2.1 Design inputs

Specify the design load. The design pressure-time curve is the 120 psi, instantaneous rise time, pressure-time curve shown in Figure 3.

- Peak pressure: $P = 120$ psi
- Duration: $T = 4,000$ ms
- Load type: rectangular load

Specify the allowable failure. The design must remain in the linear elastic range.

Specify a safety factor

$$
\Phi_p = 1.2
$$

The adjusted peak dynamic pressure from Equation 54 is

$$
P_d' = P_d \cdot \Phi_p = 120 \cdot 1.2 = 144 \text{ psi}
$$

Determine the dynamic load factor (DLF). The dynamic load factor is 2.0. The equivalent static pressure from Equation 55 is

$$
P_s = P_d' \cdot DLF = 144 \cdot 2 = 288 \text{ psi}
$$

Specify the coal mine entry geometry

Height: $H = 84$ in.
Length: $L = 240$ in.
Determine the support conditions. The seal is simply supported on four sides. It can resist translation, but it cannot resist moment at the supports.

Specify material properties

Steel yield strength: $f_y = 60,000$ psi
Concrete compressive strength: $f'_c = 5,000$ psi

Determine the dynamic increase factor

<table>
<thead>
<tr>
<th>Type of stress</th>
<th>Rebar</th>
<th>Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td>$f_{dy,m} = 1.17 \times 60,000 = 70,200$ psi</td>
<td>$f'_{dc,m} = 1.19 \times 5,000 = 5,950$ psi</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>$f_{dfa,s} = 1.10 \times 60,000 = 66,000$ psi</td>
<td>$f'_{dc,v} = 1.10 \times 5,000 = 5,500$ psi</td>
</tr>
<tr>
<td>Compression</td>
<td>$f_{dy,c} = 1.10 \times 60,000 = 66,000$ psi</td>
<td>$f'_{dc,c} = 1.12 \times 5,000 = 5,600$ psi</td>
</tr>
</tbody>
</table>

7.2.2.2 Foundation design (seal anchorage)

Determine the approximate yield line location. Solve Equation 59 for an estimate of the yield line location $x^*$. Estimate the horizontal location of the yield line by finding the real root of the cubic equation.

$$0 = 2x^3 - 6Lx^2 - 5H^2 x + \frac{15}{4}H^2$$

$$0 = 2x^3 - 6(240)x^2 - 5(84)^2 x + \frac{15}{4}(84)^2(240)$$

$$x^* = 57.174 \text{ in.}$$

Determine the approximate shear forces. Determine the approximate shear forces per foot along the perimeter at the roof, floor, and ribs of the seal using Equations 60, 61 and 62.

$$V_{sv,r}^* = \frac{3Py(2L - 2x^*)}{6L - 2x^*} = \frac{3(288)\left(\frac{84}{2}\right)[2(240) - 2(57.174)](12)}{[6(240) - 2(57.174)]}$$

$$= 120.11 \text{ kips/ft}$$
From Equation 63, select the maximum shear force per foot

\[ V_u^* = \max \{ V_{svr}^*, V_{svf}^*, V_{slh}^* \} = 120.11 \text{ kips/ft} \]

Determine the required number of rock-bolt anchors. The shear strength of a single rock-bolt anchor from Equation 64 is

\[ F_{nv} = \mu_{RB} f_{yRB} = 0.6 \cdot 60,000 = 36,000 \text{ psi} \]

Choose rock-bolt anchor diameter = 9/8 in. = #9 bar and spacing S_{RB} = 12 in.

The area of a rock-bolt anchor from Equation 65 is

\[ A_{\Phi d} = \frac{\pi \left( \frac{9}{8} \right)^2}{4} = 0.994 \text{ in.}^2 \]

The approximate number of rock-bolt anchors per foot of wall from Equation 66 is

\[ N_{\Phi d}^* = \frac{V_u^*}{F_{nv} A_{\Phi d}} = \frac{120,110}{(36,000)(0.994)} = 3.36 \approx 4 \]

Note: Round up to next integer value.

Determine the minimum seal thickness. The minimum thickness to provide adequate room and cover for the rock-bolt anchors is given by Equation 67.
\[ T_c^* = \left( N_{\Phi_d}^* - 1 \right) \cdot 12 + \Phi_d + 3 \text{ in.} = (4 - 1) \cdot 12 + 1.128 + 3 \text{ in.} = 40.128 \text{ in.} \]

7.2.2.3 Seal structure design

Estimate the reinforcing steel requirements. Estimate the horizontal and vertical positive moment capacities from Equation 68.

\[ M_{vp}^* = M_{hp}^* = \frac{P_s(x^*)^2}{5} = \frac{(288)(57.174)^2}{5} = 188,287 \text{ lb - in.} \]

The approximate distances from the extreme compression fiber to the centroid of tension reinforcement from Equations 69 and 70 are

\[ d_v^* = T_v^* - 2.5 \text{ in.} = 40.128 - 2.5 = 37.628 \text{ in.} \]

\[ d_H^* = T_v^* - 3.5 \text{ in.} = 40.128 - 3.5 = 36.628 \text{ in.} \]

The approximate area of vertical tension steel reinforcement per foot of section is determined by solving Equations 73 and 74 iteratively.

Assume \( \rho_v = 0.00192 \)

\[ A_{Vs}^* = \frac{M_{vp}^* \cdot b}{f_{dy} \cdot d_v^*} \cdot \frac{1}{1 - 0.59 \cdot \rho_v \cdot \frac{f_{dy}}{f_{dc}}} = \frac{188,287(12)}{70,200(37.628)} \cdot \frac{1}{1 - 0.59 \cdot 0.00192 \cdot \frac{70,200}{5,950}} = 0.867 \text{ in.}^2 \]

\[ \rho_v = \frac{A_{Vs}^*}{b \cdot d_v^*} = \frac{0.867}{(12) \cdot 37.628} = 0.00192 \approx 0.00192 \]

The approximate area of horizontal tension steel reinforcement per foot of section is determined by solving Equations 75 and 76 iteratively.

Assume \( \rho_H = 0.00203 \)
The approximate minimum diameters of vertical and horizontal reinforcement steel from Equations 77 and 78 are

\[
\phi_{V_{\text{min}}}^* = \sqrt{\frac{4A^*_v}{\pi}} = \sqrt{\frac{4(0.867)}{\pi}} = 1.051 \text{ in.}
\]

\[
\phi_{H_{\text{min}}}^* = \sqrt{\frac{4A^*_h}{\pi}} = \sqrt{\frac{4(0.891)}{\pi}} = 1.065 \text{ in.}
\]

Specify No. 9 rebar for vertical and horizontal reinforcement with diameter \( \phi = 1.125 \text{ in.} \)

Determine diagonal shear reinforcement

The shear strength of the concrete from Equations 79 and 80 is

\[
v_c = 2(f'_{dc})^{0.5} + 2(5,500)^{0.5} = 148 \text{ psi}
\]

\[
v_c = [1.9(f'_{dc})^{0.5} + 2500 p] \leq 3.5(f'_{dc})^{0.5} = [1.9(5,500)^{0.5} + 2,500(0.002)] = 146 \text{ psi} \leq 3.5(5,500)^{0.5} = 260 \text{ psi}
\]

Choose \( v_c = 148 \text{ psi} \).

The shear resistance of unreinforced concrete per foot of wall from Equation 81 is

\[
v_c = 12 \cdot v_c \cdot T_c^* = 12(148)(40.128) = 71.267 \text{ kips/ft}
\]
From a prior calculation, the maximum shear force per foot in the concrete is $V_u^* = 120.11$ kips/ft.

Since $V_u^*$ is greater than $V_c$, the amount of shear load that must be resisted by shear reinforcement is found from Equation 82 as

$$V_s = V_u^* - V_c = 120.11 - 71.267 = 48.84 \text{ kips}$$

The minimum spacing ($s_s$) with stirrups is 6 in. The required area of shear reinforcement with stirrups from Equation 83 is

$$A_{s,min} = \frac{(V_u^* - V_c)s_s}{d'(f_{dy})} = \frac{(48.84)6}{37.628(60,000)} = 0.130 \text{ in.}^2$$

The minimum diameter of the stirrups from Equation 84 is

$$\phi_{s,min} = \sqrt{\frac{4A_{s,min}}{\pi}} = \sqrt{\frac{4(0.130)}{\pi}} = 0.407 \text{ in.}$$

Specify #4 rebar stirrups with diameter $\phi = 0.500$ in.

**Determine the actual vertical moment capacity**

The minimum area of vertical tension reinforcement steel per foot of wall was found to be 0.867 in.²

Specify two #6 reinforcement bars with diameter $\phi = 0.75$ in. and spacing $s_v = 6$ in.

From Equation 85, the area of vertical tension reinforcement steel per foot of wall is

$$A_{v_v} = \left(\frac{\pi \cdot \phi_{v,v}}{4}\right) \cdot \frac{12}{s_v} = \left(\frac{\pi \cdot 0.75}{4}\right) \cdot \frac{12}{6} = 0.884 \text{ in.}^2 \geq 0.867 \text{ in.}^2$$

From Equation 86, the actual distance from the extreme compression fiber to the centroid of the vertical tension reinforcement is
\[ d_V = T_c - \left( CC_w + \phi_{sw} + \frac{\phi_{vw}}{2} \right) = 40.128 - \left( 1.5 + 0.5 + \frac{0.75}{2} \right) = 37.75 \text{ in.} \]

From Equation 87, the actual reinforcement ratio for the vertical tension reinforcement steel is

\[ \rho_V = \frac{A_{Vs}}{b \cdot d_V} = \frac{0.884}{12(37.75)} = 0.00195 \]

From Equations 88 and 89, the maximum allowable reinforcement ratio \( \rho_b \) is

\[ K_1 = 0.85 - 0.05 \left( \frac{f'_{dc} - 4000}{1000} \right) = 0.85 - 0.05 \left( \frac{5,950 - 4000}{1000} \right) = 0.753 \]

\[ \rho_b = \left( 0.85 \cdot K_1 \cdot \frac{f'_{dc}}{f_{dy}} \right) \left[ \frac{87000}{(87000 + f_{dy})} \right] \]

\[ = \left( 0.85 \cdot 0.753 \cdot \frac{5,950}{70,200} \right) \left[ \frac{87000}{(87000 + 70,200)} \right] = 0.03002 \]

From Equation 90, the minimum reinforcement ratio \( \rho_{s.min,V} \) is

\[ \rho_{s.min,V} = \frac{1.25 \sqrt{f'_{dc}}}{f_{dy}} = \frac{1.25 \sqrt{5,950}}{70,200} = 0.00137 \]

The actual reinforcement ratio must meet the condition given in Equation 91.

\[ \rho_{s.min,V} \leq \rho_V \leq \rho_b \]

\[ 0.00137 \leq 0.00195 \leq 0.03002 \]

The vertical ultimate moment capacity per foot of wall is found using Equations 92 and 93.

\[ a_V = \frac{A_{Vs} \cdot f_{dy}}{0.85 \cdot b \cdot f'_{dc}} = \frac{0.884 \cdot 70,200}{0.85 \cdot 12 \cdot 5,950} = 1.0225 \text{ in.} \]
\[ M_{vu} = \left( A_{Vv} \cdot \frac{f_{dv}}{b} \right) \left( d_v - \frac{a_v}{2} \right) \]

\[ = \left( 0.884 \cdot \frac{70,200}{12} \right) \left( 37.75 \cdot \frac{1.0225}{2} \right) = 192,576 \text{ lb-in.} \]

Determine the actual horizontal moment capacity

The minimum area of horizontal tension reinforcement steel per foot of wall was found to be 0.891 in.\(^2\)

Specify two #6 reinforcement bars with diameter \( \phi = 0.75 \) in. and spacing \( s_H = 6 \) in.

From Equation 94, the area of horizontal tension reinforcement steel per foot of wall is

\[ A_{hs} = \left( \frac{\pi \cdot \phi_H^2}{4} \right) \frac{12}{s_H} = \left( \frac{\pi \cdot 0.75^2}{4} \right) \frac{12}{6} = 0.884 \text{ in.}^2 \equiv 0.891 \text{ in.}^2 \]

From Equation 95, the actual distance from the extreme compression fiber to the centroid of the horizontal tension reinforcement is

\[ d_H = T_c - \left( C_{cw} + \phi_{aw} + \phi_{vw} + \frac{\phi_{hv}}{2} \right) \]

\[ = 40.128 - \left( 1.5 + 0.50 + 0.75 + \frac{0.75}{2} \right) = 37.003 \text{ in.} \]

From Equation 96, the actual reinforcement ratio for horizontal tension reinforcement steel is

\[ \rho_H = \frac{A_{hs}}{b \cdot d_H} = \frac{0.884}{12(37.00)} = 0.00199 \]

From Equations 97 and 98, the maximum allowable reinforcement ratio \( \rho_b \) is

\[ K_1 = 0.85 - 0.05 \left( \frac{f_{dc} - 4000}{1000} \right) = 0.85 - 0.05 \left( \frac{5,950 - 4000}{1000} \right) = 0.753 \]
\[ \rho_b = \left( 0.85 \cdot K_1 \cdot \frac{f'_{dc}}{f_{dy}} \right) \left( \frac{87000}{87000 + f_{dy}} \right) \]
\[ = \left( 0.85 \cdot 0.753 \cdot \frac{5,950}{70,200} \right) \left( \frac{87000}{87000 + 70,200} \right) = 0.03002 \]

From Equation 99, the minimum reinforcement ratio \( \rho_{s.min.H} \) is

\[ \rho_{s.min.H} = \frac{1.25 \sqrt{f'_{dc}}}{f_{dy}} = \frac{1.25 \sqrt{5,950}}{70,200} = 0.00137 \]

The actual reinforcement ratio must meet the condition given in Equation 100.

\[ \rho_{s.min.H} \leq \rho_{HV} \leq \rho_b \]
\[ 0.00137 \leq 0.00199 \leq 0.03002 \]

The horizontal ultimate moment capacity per foot of wall is found using Equations 101 and 102.

\[ a_H = \frac{A_{Hs} \cdot f_{dy}}{0.85 \cdot b \cdot f'_{dc}} = \frac{0.884 \cdot 70,200}{0.85 \cdot 12 \cdot 5,950} = 1.0225 \text{ in.} \]

\[ M_{Hu} = \left( A_{Hs} \cdot \frac{f_{dy}}{b} \right) \left( d_H - \frac{a_H}{2} \right) \]
\[ = \left( 0.884 \cdot \frac{70,200}{12} \right) \left( 37.00 - \frac{1.0225}{2} \right) = 188,698 \text{ lb-in. in.} \]

Determine the static properties of the design

Specify the modulus of elasticity and Poisson’s ratio for steel as 29,000,000 psi and 0.30.

From Equation 103, the modulus of elasticity for concrete is

\[ E_c = w_c^{1.5} \cdot 33 \cdot (f'_c)^{0.5} = 150^{1.5} \cdot 33 \cdot (5,000)^{0.5} = 4,287,000 \text{ psi} \]
The modular ratio from Equation 104 is

\[ n = \frac{E_s}{E_c} = \frac{29,000,000}{4,287,000} = 6.765 \]

The average reinforcement ratio from Equation 105 is

\[ \rho = \frac{(\rho_v + \rho_h)}{2} = \frac{(0.00195 + 0.00199)}{2} = 0.00197 \]

The moment of inertia from Equation 106 is

\[ I_g = \frac{T_c^3}{12} = \frac{(40.128 \text{ in.})^3}{12} = 5,385 \text{ in.}^4 \]

Using Figure 99 with \( \rho = 0.00197 \) and \( n = 6.765 \), the cracked concrete coefficient (\( F \)) is determined.

\[ F = 0.01 \]

The cracked moment of inertia from Equation 107 is

\[ I_c = F \cdot \left( \frac{d_v + d_a}{2} \right)^3 = 0.01 \cdot \left( \frac{37.75 + 37.00}{2} \right)^3 = 522.1 \text{ in.}^4 \]

The averaged moment of inertia from Equation 108 is

\[ I_a = \frac{I_g + I_c}{2} = \frac{5,385 + 522.1}{2} = 2,953.6 \text{ in.}^4 \]

The flexural rigidity of the slab from Equation 109 is

\[ D = \frac{E_c \cdot I_a}{1 - v^2} = \frac{4,287,000 \cdot 2,953.6}{1 - 0.17^2} = 1.304 \times 10^{10} \text{ lb-in.} \]

The height-to-length ratio of the seal structure is

\[ \frac{H}{L} = \frac{84}{240} = 0.350 \]
Using Figure 100, the moment and deflection coefficients, $\beta_{1V}$, $\beta_{1H}$, and $\gamma_1$, for a two-way wall with all edges simply supported are determined.

\[ \beta_{1V} = 0.12 \]
\[ \beta_{1H} = 0.042 \]
\[ \gamma_1 = 0.012 \]

Determine the actual yield line location and the ultimate resistance

The actual moment capacities found earlier are $M_{vp} = 192,576$ lb-in. and $M_{hp} = 188,698$ lb-in. Equations 110 and 111 are solved simultaneously for the ultimate resistance $r_u$ and the actual yield line location $x$.

\[ r_u = \frac{M_{vp} (6L - 2x)}{H (L^2)} \cdot (3L - 4x) \]
\[ r_u = \frac{5M_{hp}}{x^2} \]

The system of equations reduces to

\[ 0 = \left( 2M_{vp} \right) x^3 - \left( 6LM_{vp} \right) x^2 - \left( 5H^2 M_{hp} \right) x + \frac{15}{4} H^2 LM_{hp} \]

\[ 0 = 2(192,576)x^3 - 6(240)(192,576)x^2 - 5(84)^2 (188,698)x + \frac{15}{4} (84)^2 (240)(188,698) \]

\[ 0 = 385,152 \cdot x^3 - 277,309,440 \cdot x^2 - 6,657,265,440 \cdot x + 1,198,000,000,000 \]

\[ x = 56.67734 \approx 56.7 \text{ in.} \]

The ultimate elastic resistance is
\[ r_u = \frac{(M_{vp})(6L-2x)}{\left(\frac{H}{2}\right)^2(3L-4x)} = \frac{(192,576)(6 \cdot 240 - 2 \cdot 56.7)}{(84)^2(3 \cdot 240 - 4 \cdot 56.7)} = 293.6 \text{ psi} \]

\[ r_u = \frac{5M_{hp}}{x^2} = \frac{5 \cdot 188,698}{56.7^2} = 293.5 \text{ psi} \]

The elastic deflection of the seal at ultimate elastic resistance from Equation 112 is

\[ X_e = \frac{Y_1 \cdot r_u \cdot H^4}{D} = \frac{0.012 \cdot 293.5 \cdot 84^4}{1.304 \times 10^{10}} = 0.0134 \text{ in.} \]

The elastic stiffness of the seal from Equation 113 is

\[ K_E = \frac{r_u}{X_e} = \frac{293.5}{0.0134} = 21,902 \text{ psi/in.} \]

Determine the direct shear capacity of concrete

From Equation 114, the direct shear capacity of the concrete near the supports is

\[ V_d = 0.16 f'_d \cdot b \cdot d_H = 0.16 \cdot 5,500 \cdot 12 \cdot 37.00 = 390,720 \text{ lb/ft of wall} \]

The actual shear forces per foot of wall at the roof, floor, and ribs are found with Equations 115, 116 and 117.

\[ \text{Shear at roof } V_{s,f} = \frac{3P_s y(2L-2x) \cdot 12}{(6L-2x)} = \frac{3 \cdot 288 \cdot 84 / 2 \cdot (2 \cdot 240 - 2 \cdot 56.7) \cdot 12}{(6 \cdot 240 - 2 \cdot 56.7)} = 120,336 \text{ lb/ft of wall} \]
Shear at floor \( V_{sVf} = \frac{3P_s(H - y)(2L - 2x)\cdot 12}{(6L - 2x)} \)

\[ = \frac{3 \cdot 288(84 - 84/2)(2 \cdot 240 - 2 \cdot 56.7)\cdot 12}{(6 \cdot 240 - 2 \cdot 56.7)} \]

\[ = 120,336 \text{ lb/ft of wall} \]

Shear at ribs \( V_{sH} = \frac{3P_x \cdot 12}{5} = \frac{3 \cdot 288 \cdot 56.7 \cdot 12}{5} = 117,573 \text{ lb/ft of wall} \)

The maximum shear force per foot of wall from Equation 118 is

\[ M_{\text{maximum shear}} V_u = \max \{V_{sVf}, V_{sVr}, V_{sH}\} = 120,336 \text{ lb/ft of wall} \]

The direct shear capacity of the concrete is greater than the shear forces in the concrete.

\[ V_d \geq V_u \]

390,720 \( \geq \) 120,336

Determine the dynamic response of the seal

The length-to-height ratio of the seal structure is

\[ \frac{L}{H} = \frac{240}{84} = 2.857 \]

From Table 28, the elastic load mass factor for a two-way wall element with all sides simply supported is

\[ K_{LM_{\text{elastic}}} = 0.79 \]

The yield line location to length ratio of the seal structure is

\[ \frac{x}{L} = \frac{56.7}{240 \text{ in.}} = 0.236 \]
From Figure 101, the plastic load mass factor for a two-way wall element with all sides simply supported is

\[ K_{LM_{\text{plastic}}} = 0.61 \]

The effective load mass factor from Equation 120 is

\[ K_{LM} = \frac{K_{LM_{\text{plastic}}} + K_{LM_{\text{elastic}}}}{2} = \frac{(0.79 + 0.61)}{2} = 0.70 \]

The effective unit mass from Equation 121 is

\[ m_e = \frac{\rho c \cdot T_e \cdot K_{LM}}{g} = \frac{150 \text{ lb} \cdot (1\text{ ft})^3}{32.2 \text{ ft} \cdot \frac{12\text{ in.}}{1\text{ ft}} \cdot 1\text{ sec} \cdot \frac{1\text{ sec}}{1000\text{ ms}}} \cdot 40.128\text{ in.} \cdot 0.70 = 6,310.4 \frac{\text{psi-m}^2}{\text{in.}} \]

The natural period of vibration from Equation 122 is

\[ T_N = 2\pi \left(\frac{m_e}{K_E}\right)^{0.5} = 2\pi \left(\frac{6,310.4}{21,902}\right)^{0.5} = 3.37\text{ ms} \]

Response chart parameters are calculated as follows.

\[ \frac{r_u}{P_d'} = \frac{293.5\text{ psi}}{144\text{ psi}} = 2.038 \text{ and } \frac{T}{T_N} = \frac{4,000\text{ ms}}{3.37\text{ ms}} = 1,187 \]

From Figure 102, the ratio \(X_m/X_e\) is found as 1.00. From Equation 123, the maximum dynamic displacement is

\[ x_m = x_e \cdot \frac{X_m}{X_e} = 0.0134 \cdot 1.00 = 0.0134\text{ in.} \]

From Equation 53, the maximum dynamic displacement is less than or equal to the maximum elastic displacement, and therefore, the design is elastic.

\[ X_{\text{max}} \leq X_{\text{elastic}} \]
7.3 Design charts for reinforced concrete seals

7.3.1 RC seal for 120-psi pressure-time curve with an instantaneous rise time

A large number of reinforced concrete seal designs were completed for typical coal mine entry sizes using the procedure discussed in the previous sections. This set of reinforced concrete seal designs are for the 120-psi pressure-time curve with an instantaneous rise time, as shown in Figure 3. Parameters used in these designs are concrete with a compressive strength of 5,000 psi and steel reinforcement with Grade 60 bar, 6 in. on centers, both faces, and both directions, vertically and horizontally. The seal structure is attached to the surrounding rock with rock-bolt anchors that are No. 9 bar, Grade 60, and 12 in. on centers.

Figure 103 presents design charts for reinforced concrete seals to withstand the 120-psi pressure-time curve with an instantaneous rise time. The upper chart gives the minimum rebar diameter to resist the applied load through elastic flexure for three different entry widths, i.e., 16, 20 and 24 ft. The lower chart gives the minimum seal thickness for three different entry widths. In this example design chart, the seal foundation design always controls the required seal thickness. The required minimum seal thickness is governed by the required number of rock-bolt anchors at the rock-seal interface.

To illustrate the use of this design chart for the 120-psi pressure-time curve with an instantaneous rise time, assume that the coal mine entry dimensions are 20 ft wide and 9 ft high. From the upper chart, the internal reinforcement diameter is 0.875 in., or No. 7 bar. These bars are spaced 6 in. center-to-center, on both faces, and in both directions, vertically and horizontally. From the lower chart, the minimum seal thickness is 52 in. The seal requires five rows of No. 9 bar rock-bolt anchors on 1-ft centers.

7.3.2 RC seal for the 120-psi pressure-time curve with a 0.25-sec rise time

This set of reinforced concrete seal designs are for the 120-psi pressure-time curve with a 0.25-sec rise time, as shown in Figure 4. Because of the slow rise time, the dynamic load factor used in these designs is 1.0. The parameters used in these designs are the same as the prior example, i.e., concrete with a compressive strength of 5,000 psi and steel reinforcement
Figure 103. Reinforced concrete mine seal design chart for a 120-psi pressure-time curve with an instantaneous rise time.
with Grade 60 bar, 6 in. on centers, both faces, and both directions, vertically and horizontally. The seal structure is attached to the surrounding rock with rock-bolt anchors that are No. 9 bar, Grade 60, and 12 in. on centers.

Figure 104 presents design charts for reinforced concrete seals to withstand the 120-psi pressure-time curve with a 0.25-sec rise time. To illustrate the use of this design chart, assume that the coal mine entry dimensions are 20 ft wide and 9 ft high, as before. From the upper chart, the internal reinforcement diameter is 1.125 in. or No. 9 bar. These bars are spaced 6 in. center-to-center, on both faces, and in both directions, vertically and horizontally. From the lower chart, the minimum seal thickness is 28 in. The seal requires three rows of No. 9 bar rock-bolt anchors on 1-ft centers.

### 7.4 Design of unreinforced concrete plug seals

The design of unreinforced concrete plug seals follows the general outline shown in Figure 98, with the following detailed steps.

- **A. Design inputs**
  1. Specify design load
  2. Specify safety factor
  3. Determine dynamic load factor (DLF)
  4. Specify coal mine geometry
  5. Specify material properties
  6. Determine dynamic increase factor (DIF)

- **B. Foundation design (seal anchorage)**
  1. Determine approximate shear forces
  2. Determine number of rock-bolt anchors
  3. Determine minimum seal thickness based on foundation design

- **C. Seal structure design**
  1. Determine minimum seal thickness based on flexure analysis
  2. Determine minimum seal thickness based on shear analysis
Figure 104. Reinforced concrete mine seal design chart for a 120-psi pressure-time curve with a 0.25-sec rise time.

Design Chart for RC Seal
120 psi 0.25 second rise time

[Diagram showing design chart with dimensions and rebar requirements for different opening heights and seal thicknesses.]
These detailed steps are illustrated in the following flowchart for reinforced concrete seal design. The procedures follow the design manual for concrete, ACI 318-08 (ACI 2008).

7.5 Procedure for the design of unreinforced concrete plug seals

7.5.1 Specify design load

The design load is found by choosing one of the design pressure-time curves specified in the Final Rule (2008) shown in Figures 1 through 4 and 2 of this report.

7.5.2 Specify safety factor

This design procedure uses a safety factor similar to that for reinforced concrete seal design. The equivalent TNT charge weight, hence the peak dynamic pressure, is increased by a factor of 1.2 (UFC 3-340-02, DOD 2008). The adjusted dynamic pressure is found as

$$P_{d'} = P_d \cdot \phi_p$$  \hspace{1cm} (122)

where $P_d$ is the peak dynamic pressure and $P_{d'}$ is the adjusted dynamic pressure.

Determine the dynamic load factor (DLF)

The DLF transforms the dynamic design problem into an equivalent static design problem, as discussed in Chapter 3-2 of this report. The DLF for the pressure-time curves in the Final Rule (2008) are either 2.0 or 1.0 as presented in Tables 3 and 4. The equivalent static design pressure is

$$P_s = P_{d'} \cdot DLF$$  \hspace{1cm} (123)

Specify the coal mine entry geometry

The width, $W$, for a typical coal mine entry is 20 ft, but may range from about 16 to 24 ft in practice. The height, $H$, of a coal mine entry may range from about 3 to 17 ft, and is typically about 7 ft.
Specify material properties

The design of an unreinforced concrete plug seal requires the compressive strength of concrete ($f'_c$) and the yield strength of rebar and rock bolts ($f_y$). The concrete compressive strength shall not be less than 2,500 psi, as provided in ACI 318-08 (2008) Section 1.1.1.

Determine the dynamic increase factor (DIF)

As discussed in Chapter 3-2 of this report, construction materials subject to dynamic loading typically exhibit higher strength than materials subject to ordinary static loading, which can be up to about 20%. In these designs, no adjustment is made; the DIF is always 1.0.

Foundation design (seal anchorage)

Determine the approximate shear forces

When the vertical and horizontal moment capacities are equal, the shear forces acting at the different edges of the seal depend on the seal geometry only, and therefore, the foundation design for an unreinforced concrete plug seal is the same as for a reinforced concrete seal. Using Equation 59, the approximate location of the yield line is estimated. Next, using Equations 60, 61, and 62, the approximate shear forces at the roof, floor, and ribs are calculated. Finally, Equation 63 is used to determine the maximum shear force per foot of wall, $V_u^*$.

Using this method, the maximum shear force per foot of wall is determined for a wide range of typical coal mine seal dimensions. Table 29 presents the maximum shear force for two different equivalent static pressures, i.e., $P_s = 288$ and $P_s = 144$ psi.

An estimate of the maximum shear force per foot of wall can be obtained from the average shear force per foot of wall as follows.

$$V_{u}^{a} = P_s \frac{\text{area}}{\text{perimeter}} = P_s \frac{W \cdot H}{2 \cdot W + 2 \cdot H}$$

(124)
where

\[ P_s = \text{equivalent static design pressure} \]
\[ W = \text{seal width} \]
\[ H = \text{seal height} \]

Note that Equation 124 calculates an average shear force around the seal perimeter, and it may not be conservative. Maximum shear force calculated using the more exact yield line method discussed above and presented in Table 29 can be up to 20% larger than the simple average method using Equation 124.

### Table 29. Maximum shear force per foot of wall, \( V_u \), lb.

| Height (ft) | Length = 16 ft | | | Length = 20 ft | | | Length = 24 ft | | |
|-------------|----------------|---|----------------|---|---|---|---|---|
| Height (ft) | \( P_s = 288 \text{ psi} \) | \( P_s = 144 \text{ psi} \) | \( P_s = 288 \text{ psi} \) | \( P_s = 144 \text{ psi} \) | \( P_s = 288 \text{ psi} \) | \( P_s = 144 \text{ psi} \) | \( P_s = 288 \text{ psi} \) | \( P_s = 144 \text{ psi} \) |
| 4           | 72,483         | 36,241         | 74,487         | 37,244         | 75,851         | 37,925         | 74,487         | 37,244         |
| 5           | 87,564         | 43,782         | 90,603         | 45,302         | 92,687         | 46,343         | 90,603         | 45,302         |
| 6           | 101,564        | 50,782         | 105,796        | 52,898         | 108,724        | 54,362         | 105,796        | 52,898         |
| 7           | 114,558        | 57,279         | 120,111        | 60,056         | 123,991        | 61,996         | 120,111        | 60,056         |
| 8           | 126,625        | 63,313         | 133,597        | 66,798         | 138,521        | 69,260         | 133,597        | 66,798         |
| 9           | 138,046        | 69,023         | 146,303        | 73,152         | 152,346        | 76,173         | 146,303        | 73,152         |
| 10          | 148,883        | 74,442         | 158,282        | 79,141         | 165,500        | 82,750         | 158,282        | 79,141         |
| 11          | 158,941        | 79,470         | 169,727        | 84,863         | 178,019        | 89,009         | 169,727        | 84,863         |
| 12          | 168,263        | 84,131         | 180,805        | 90,403         | 189,938        | 94,969         | 180,805        | 90,403         |

Determine the number of rock-bolt anchors

This calculation is identical to that presented earlier for reinforced concrete seal design.

The shear strength of a single rock-bolt anchor is

\[ F_{nv} = \mu_{RB} f_{yRB} \]  \hspace{1cm} (125)

where \( f_{yRB} \) is the yield strength of the rock-bolt anchor steel and \( \mu_{RB} \) is the shear strength reduction factor taken as 0.6.

The area of a rock-bolt anchor with diameter \( \phi_d \) is
\[ A_{\phi_d} = \frac{\pi \phi_d^2}{4} \]  \hspace{1cm} (126)

The approximate number of rock-bolt anchors required of diameter \( \phi_d \) per foot of wall is

\[ N_{\phi_d}^* = \frac{V_u^*}{F_{nu} A_{\phi_d}} \]  \hspace{1cm} (127)

Note: Round up to the next integer value.

**Determine the minimum seal thickness based on foundation design**

The minimum seal thickness, \( T_c^* \) must provide adequate room for a 12-in. center-to-center rock-bolt anchor spacing plus a 1.5-in. clear cover on both sides.

\[ T_c^* = (N_{b_j}^* - 1) \cdot 12 + \phi_d + 3 \text{ in.} \]  \hspace{1cm} (128)

In unreinforced concrete plug seal design, the seal thickness is usually not controlled by the foundation design. To reduce the risk of concrete breakout, it is preferable to maximize the concrete cover depth. Therefore, the location of the center row for an odd number of rows, or the two center rows for an even number of rows, should be at or close to the mid-thickness of the plug. The rock-bolt anchors are embedded 12 in. into the concrete.

**Seal structure design**

**Determine the minimum seal thickness based on a flexure analysis**

The ratio of the spans for a seal (long span/short span) usually exceeds 2 in most practical situations. Hence, an unreinforced concrete plug seal resists moment in the shorter span essentially as a one-way slab. For design purposes, a one-way slab is assumed to act as a series of parallel, independent 1-ft-wide strips of slab, continuous over the supports. This series of parallel beams can be designed by following the plain concrete requirements in Chapter 22 of ACI 318-08 (ACI 2008).
The flexural design of a beam follows ACI 318-08 Section 22.5.1. The nominal moment capacity, $M_n$, must exceed the ultimate moment, $M_u$.

$$\phi M_n \geq M_u$$ (129)

where $\phi = \text{moment capacity reduction factor}$.

The factor $\phi$ is usually set at 0.9 for tension controlled sections. In this application, it is set at 1.0 because of the 1.2 increase factor applied to the load.

The nominal moment capacity is calculated as

$$M_n = 5S_m\sqrt{f'_c}$$ (130)

where $S_m$ is the elastic section modulus and $f'_c$ is the compressive strength of concrete.

The elastic section modulus is calculated from the geometry as

$$S_m = \frac{bT^2}{6}$$ (131)

where $b$ is the unit width and $T$ is the seal thickness.

Assuming simple support conditions, the ultimate moment is calculated as

$$M_u = \frac{P_s \cdot b \cdot H^2}{8}$$ (132)

where $P_s$ is the equivalent static design pressure and $H$ is the seal height.

Combining Equations 130, 131, and 132 and solving for the thickness, $T$, gives

$$T \geq H\sqrt{\frac{3P_s}{20\cdot f'_c}}$$ (133)
The term deep beam is defined in ACI 318-08 Section 10.7.1 as a member loaded on one face and supported on the opposite face and having a clear span, \( l_n \), equal to or less than four times the overall member height, \( h \). ACI 318-08 Section 11.8.2, Deep Beams, gives essentially the same definition. Both sections require deep beam designs using non-linear analyses or Strut-and-Tie Models. Unreinforced concrete plug seals always fit the definition for a deep beam. However, plug seals are not supported at the unloaded face. Instead, they are supported along the length of the seal edges, i.e., the seal thickness. A Strut-and-Tie Model that accurately represents these support conditions does not exist; therefore, conventional beam analysis is applied.

**Determine the minimum seal thickness based on shear analysis**

The shear design of a seal follows ACI 318-08 Section 22.5.4. The nominal shear capacity, \( V_n \), must exceed the ultimate shear load, \( V_u \).

\[
\phi V_n \geq V_u
\]  

(134)

where \( \phi = \) shear capacity reduction factor.

The factor \( \phi \) is usually set at 0.75 for shear-controlled sections. In this application, it is set at 1.0 because of the 1.2 increase factor applied to the load.

For two-way action, the nominal shear capacity is

\[
V_n = \left[ \frac{4}{3} + \frac{8}{3\beta} \right] b \cdot T \cdot \sqrt{f_{c'}^\prime} \leq 2.66 \cdot b \cdot T \cdot \sqrt{f_{c'}^\prime}
\]  

(135)

where

\( \beta = \) span ratio (L/H)  
\( b = \) unit width  
\( T = \) seal thickness  
\( f_{c'}^\prime = \) compressive strength of concrete

The ultimate shear load, \( V_u \), is the same shear load as that calculated for the foundation design. It is calculated by the solution of Equations 59
through 63 or it can be taken directly from Table 29. Alternatively, its average value can be estimated using Equation 126. Note that this average value may not be conservative, and it is always less that those values presented in Table 29.

Substituting Equation 137 into Equation 136 and solving for $T$ yields the following expressions for $T$.

$$T \geq \frac{V_u}{\frac{4}{3} + \frac{8}{3\beta}} \cdot \beta \cdot \sqrt{f'_c}$$

(136)

$$T \geq \frac{V_u}{2.66 \cdot \beta \cdot \sqrt{f'_c}}$$

(137)

Note: Choose the larger value for the thickness required to resist the shear load.

7.6 Design example for an unreinforced concrete plug seal

The following numerical example illustrates the design procedure for the unreinforced concrete plug seals presented in Section 7-5-1 of this report.

7.6.1 Specify design load

The design pressure-time curve is the 120-psi, instantaneous rise time, pressure-time curve shown in Figure 3.

Peak pressure: $P = 120$ psi
Duration: $T = 4,000$ ms
Load type: rectangular load

7.6.2 Specify a safety factor

$$\Phi_p = 1.2$$

The adjusted peak dynamic pressure from Equation 122 is

$$P'_d = P_d \cdot \Phi_p = 120 \cdot 1.2 = 144 \text{ psi}$$
Determine the dynamic load factor (DLF)

The dynamic load factor is 2.0. The equivalent static pressure from Equation 125 is

$$P_s = P_d' \cdot DLF = 144 \cdot 2 = 288 \text{ psi}$$

Specify the coal mine entry geometry

Height: $H = 84$ in.
Length: $L = 240$ in.

Specify material properties

Steel yield strength: $f_y = 60,000$ psi
Concrete compressive strength: $f_c' = 2,500$ psi

Determine the dynamic increase factor

The dynamic increase factor for both steel and concrete is 1.0.

Foundation design (seal anchorage)

Determine the approximate shear forces

From Table 29, the maximum shear force per foot of wall is

$$V_u^* = 120,111 \text{ lb per ft}$$

Alternatively, the average shear force per foot of wall from Equation 126 is

$$V_u^a = P_s \cdot \frac{\text{area}}{\text{perimeter}} = P_s \cdot \frac{W \cdot H}{2 \cdot W + 2 \cdot H}$$

$$= 288 \cdot 144 \cdot \frac{20.7}{2 \cdot 20 + 2 \cdot 2.7} = 107,520 \text{ lb per ft}$$

Use $V_u^* = 120,111 \text{ lb per ft}$

Determine the number of rock-bolt anchors
The shear strength of a single rock-bolt anchor from Equation 127 is

\[ F_{nv} = \mu_{RB} f_{yRB} = 0.6 \cdot 60,000 = 36,000 \text{ psi} \]

where \( \mu_{RB} \) is the coefficient of friction. For concrete that is cast against hardened concrete not roughened, the shear resistance is primarily due to the rock-bolt anchors (ACI 318-08, Section 11.6.4.3).

Specify No. 9 bar rock-bolt anchors with a diameter of 1.125 in. and a spacing \( S_{RB} = 12 \text{ in.} \).

The area of a rock-bolt anchor from Equation 128 is

\[ A_{\phi} = \frac{\pi \phi^2}{4} = \frac{\pi \cdot 1.125^2}{4} = 0.994 \text{ in.}^2 \]

The approximate number of rock-bolt anchors needed from Equation 127 is

\[ N_{\phi u}^* = \frac{V_{u}^*}{F_{nv} A_{\phi}} = \frac{120,111}{36,000 \cdot 0.994} = 3.36 \approx 4 \]

Note: Round up to the next integer value.

**Determine the minimum seal thickness based on foundation design**

The minimum seal thickness needed to encase the required number of rock-bolt anchors in the foundation is found with Equation 130.

\[ T_{c}^* = (N_{\phi u}^* - 1) \cdot 12 + \phi_d + 3 \text{ in.} = (4 - 1) \cdot 12 + 1.125 + 3 \text{ in.} = 40.125 \text{ in.} \]

**Seal structure design**

**Determine the minimum seal thickness based on a flexure analysis**

The minimum seal thickness based on a flexure analysis from Equation 135 is
Determine minimum seal thickness based on shear analysis

The minimum seal thicknesses based on a shear analysis from Equations 136 and 137 are

\[ T \geq H \frac{3P_a}{20 \cdot \sqrt{f_c}} = 84 \frac{3.288}{20 \cdot \sqrt{2,500}} = 78.08 \text{ in.} \approx 80 \text{ in.} \]

Note that \( T \) is rounded up to the next even dimension.

Choose 90 in. as the thickness required to resist the shear load in the concrete.

For the final design, choose the largest of (1) the minimum seal thickness needed to encase the rock-bolt anchors, \( T_c^* = 40.125 \text{ in.} \), (2) the minimum seal thickness based on a flexure analysis, \( T = 80 \text{ in.} \), or (3) the minimum seal thickness based on a shear analysis, \( T = 90 \text{ in.} \). In this case, the minimum thickness required to resist the shear load in the concrete dictates the design. The required minimum seal thickness is 90 in.

### 7.7 Design charts for unreinforced concrete plug seals

#### 7.7.1 Unreinforced concrete plug seal for the 120-psi pressure-time curve with an instantaneous rise time

A large number of unreinforced concrete plug seal designs were completed for typical coal mine entry sizes using the procedure discussed in the previous sections. Figure 106 shows the design chart for unreinforced concrete plug seals to resist the 120-psi pressure-time curve with an instantaneous rise time, as shown in Figure 3. These designs use concrete with a compressive strength of 2,500 or 5,000 psi. The seal structure is anchored to the surrounding rock with rock-bolt anchors that are No. 9 bar, Grade 60, and 12 in. on centers, as shown in Figure 105.
Figure 105. Typical cross section for an unreinforced concrete plug seal design.

Figure 106 shows the design chart for unreinforced concrete plug seals. The lower plot for the seal foundation design gives the number of rows of rock-bolt anchors for various coal mine entry heights and widths. The upper plot gives the required minimum seal thickness for various coal mine entry heights and widths for two different concrete compressive strengths. With unreinforced concrete plug seals, the required thickness is usually governed by the shear strength of the concrete, and to a lesser extent, the entry width.

To illustrate the use of this design chart for the 120-psi pressure-time curve with an instantaneous rise time, assume that the coal mine entry dimensions are 20 ft wide and 7 ft high. From the lower chart, four rows of rock-bolt anchors are required. From the upper chart, the required seal thickness is 88 in. with 2,500-psi concrete and 66 in. with 5,000-psi concrete.
Figure 106. Unreinforced concrete plug seal design chart for the 120-psi pressure-time curve with an instantaneous rise time.
7.7.2 Unreinforced concrete plug seal for the 120-psi pressure-time curve with a 0.25-sec rise time

This set of unreinforced concrete plug seal designs are for the 120-psi pressure-time curve with 0.25-sec rise time, as shown in Figure 4. Because of the slow rise time, the dynamic load factor used in these designs is 1.0. These designs use concrete with compressive strengths of 2,500 or 5,000 psi. The seal structure is anchored to the surrounding rock with rock-bolt anchors that are No. 9 bar, Grade 60, and 12 in. on centers, as shown in Figure 105.

Figure 107 shows the design chart for unreinforced concrete plug seals to resist the 120-psi pressure-time curve with a 0.25-sec rise time. To illustrate the use of this design chart for the 120-psi pressure-time curve with an instantaneous rise time, assume that the coal mine entry dimensions are 20 ft wide and 7 ft high, as before. From the lower chart, two rows of rock-bolt anchors are required. From the upper chart, the required seal thickness is 55 in. with 2,500-psi concrete and 46 in. with 5,000-psi concrete.
Figure 107. Unreinforced concrete plug seal design chart for the 120-psi pressure-time curve with a 0.25-sec rise time.
8 Behavior of 120-psi Seals Subject to Methane-Air Detonation Pressure

A reinforced concrete mine seal must be designed to have a static ultimate resistance of 288 psi in order to respond elastically to the 120-psi instantaneous rise time pressure-time curve in the Final Rule (2008). The designs created in Chapter 7 satisfy these strength requirements. However, this design pressure-time curve is a simplified curve and not an actual methane-air detonation pressure-time curve. The following sections analyze how the reinforced concrete designs presented in Chapter 7 will respond to an actual methane-air detonation pressure-time curve.

8.1 Potential for an explosion pressure greater than 120 psi

Although the 120-psi instantaneous rise time pressure-time curve in the Final Rule (2008) is a simplified rectangular curve with a peak pressure of 120 psi, many sources have shown theoretically, computationally, and experimentally that actual methane-air detonation pressures can be much higher, especially near the beginning of the pressure-time history (Zipf et al. 2007; McMahon et al. 2010; Sapko et al. 2010; Zipf et al. 2011; Wolanski et al. 1981). After attenuation of the initial shock pressure from a methane-air explosion, the ideal gas law dictates that the gas pressure will remain around 120 psi, called the constant volume explosion pressure, for as long as the volume remains constant and unvented (Zipf et al. 2007). The 120-psi pressure-time curve in the Final Rule (2008) appears to be based on the constant volume explosion pressure for methane-air. However, this pressure-time curve ignores the short duration shock pressure at the beginning of the pressure-time history. The theoretical Chapman-Jouguet detonation pressure is about 240 psi, and for many seal locations, a detonation wave could impact the seal at normal incidence, leading to an even higher reflected wave pressure of about 640 psi (Zipf et al. 2007). Additionally, multiple pressure waves travelling by different routes could arrive at the same seal at the same time, further increasing the pressure that seals must withstand.

The time duration of the pressure-time curve is also important when conducting a dynamic analysis. The load duration of four seconds in the Final Rule (2008) ensures a worst case response because the ratio of load
duration to natural period is extremely large for any practical seal structure. Methane-air detonation pressures have been measured in experiments. If the highest measured pressure is assumed to be reflected against a rigid seal within an idealized mine tunnel, it should represent the “worst case” pressure that a coal mine seal must resist. However, the reflected waveforms have not actually been measured directly to date. Computational models have therefore been used to calculate the reflected pressure-time curve from the measured static (side-on) pressure-time curve for an ideal methane-air detonation wave impacting a coal mine seal.

8.2 Measured and computed static pressure-time curves for methane-air detonations

Figure 108 shows typical measured pressure-time curves from experiments over a wide range of methane-in-air concentrations. The test set-up was comparable to a coal mine tunnel and provided results relevant to the design of coal mine seals. These measured curves are of the static (side-on) pressure. They are not the normally reflected pressure. Furthermore, the hot explosion gases were allowed to vent, and thus these experiments did not capture the long-duration, constant volume, explosion pressure because of the venting. The worst case from the three tests shown in Figure 108 is Test 36 with a peak pressure of 617.72 psi as shown in Figure 109. Methane can detonate over a wide range of mixture ratios with air (Zipf et al. 2011). The pressure-time curve for Test 29 in Figure 108 shows that, even when methane concentrations are in the low range, the detonation pressures are higher than the 120-psi design pressure prescribed in the Final Rule (2008).

The measured pressure-time curves shown in Figures 108 and 109 from methane-air detonation experiments are static (side-on) pressures and are not normally reflected pressures. Computational modeling must be used to calculate the normally reflected pressure-time curve based on the measured static pressure-time curve. In support of studies of blast-wave attenuators for coal mine seals (Sapko et al. 2010), Britt calculated static (side-on) and normal-reflections for methane-air detonation pressures using the SAGE computer model (Gittings et al. 2005). Of particular use to the present analysis is Britt’s computation of pressures reflecting on a normal surface from the detonation of a stoichiometric mix of methane and air that completely fills a smooth walled tunnel.
Figure 108. Measured static (side-on) pressure for a methane-air detonation with lean (7.3%), rich (14%), and stoichiometric (10.2%) mixtures. The time scale shows only the initial 5 msec of the pressure-time curves (Zipf et al. 2011).

The computed static (side-on) pressure-time curve created by Britt was validated by comparing it to the experimentally measured, static pressure-time curves. Figure 110 shows that the experimental and computed pressure-time curves are very similar. The most significant difference is that the experimental pressures eventually return to zero due to venting, whereas the computed static (side-on) pressures, without venting, remain around the constant volume explosion pressure of 120 psi.
8.3 Computed normally reflected pressure-time curve for methane-air detonations

From the computed static pressure-time curve, further calculations yield a computed normally reflected pressure-time curve, as shown in Figure 111. This curve is similar in shape to the computed and measured static (side-on) pressure-time curves, but its magnitude is two to three times greater due to its reflection from a normal surface. Normally reflected pressure divided by the static pressure gives the reflection factor. The computed normally reflected and static pressures, along with the reflection factor at each point, are shown in Figure 112. After about 100 msec, the reflection
factor is one, indicating no reflection. Only the initial shock pressure is reflected, and the later, long-duration static pressure is not. The computed normally reflected pressure is not permitted to vent, and it too converges to a constant volume explosion pressure of about 120 psi after the initial shock pressure attenuates.

Figure 110. Comparison of measured and computed static (side-on) pressure for methane-air detonation at different compositions.
Figure 11.1. Computed normally reflected pressure-time curve for detonation of stoichiometric methane-air mixture.

Figure 113 shows that the normally reflected pressure contains substantially more energy than the static (side-on) pressure, as indicated by the difference in impulse. Figure 113 also shows that both the static and normally reflected pressure-time curves converge after about 100 msec of duration, while the impulse curves remain at a nearly constant difference for the remainder of the pressure-time history.

The extreme pressure spike of over 2,250 psi seen at the beginning of the pressure-time curve is not the traditional peak pressure value but is an artifact of the pressure measurement coming from within the detonation wave. Conventional blast pressure profiles are a measure of the shock
wave traveling through air. In a case where the methane-air mixture is in contact with the mine seal, the pressure on the mine seal comes from the detonation of a fuel cloud that is in direct contact with the face of the seal.

Figure 112. Comparison of a computed normally reflected pressure-time curve of detonation wave with measured static (side-on) pressure-time curve for detonation of a stoichiometric methane-air mixture. Also shown is the reflection factor, which is the ratio between the normally reflected and the static pressure-time curve.

This is essentially equivalent to measuring the detonation pressure from within an explosive substance.

Although the initial spike appears very large, it only has a small effect on the mine seal response due its short duration. Figure 114 is a short time-scale view of the initial pressure spike and its associated impulse. The pressure spike is over after about 0.30 milliseconds, and at that time the
The experimental pressures from Zipf et al. (2011) and the computational pressures from Britt clearly demonstrate a concurrence between the measured and theoretical values. The quasi-static (side-on) pressure-time histories (Figure 110) show the expected 240-psi Chapman-Jouguet detonation pressure at the apparent intersection of the pressure-time curve with the initial pressure spike. Similarly, for the computed, normally reflected pressure-time curve, the theoretical maximum reflected wave
pressure of 640 psi can be clearly seen intersecting the initial pressure spike history in Figure 111 and in Figure 115 (time scale windowed around peak). The constant volume explosion pressure of 120 psi, which is theoretically predicted by the ideal gas law equation, is observed in both the static (side-on) and normally reflected pressure-time curves shown in Figures 110 and 111. The theoretical, experimental, and computational pressures agree well, providing confidence in the accuracy of all three values. The computed normally reflected pressure-time curve by Britt, derived from the measured static (side-on) pressure-time curve, provides a realistic “worst case” scenario for analyzing mine seal response to methane-air detonations.

Figure 114. Short time-scale view of the initial pressure spike of the normally reflected pressure and impulse from the detonation of a stoichiometric methane-air mixture.
Figure 115. Computed normally reflected pressure and impulse histories for the detonation of a stoichiometric, methane-air mixture at a short time scale focused at the origin.

8.4 Response of 120-psi seals to a methane-air detonation pressure as calculated by WAC

A sample of representative reinforced concrete seal designs, all of which are likely to produce significant displacement response when subjected to a normally reflected pressure-time curve, were selected from the design charts presented in Chapter 7. These designs were analyzed using WAC (Slawson 1995), which is a Single Degree of Freedom (SDOF) analysis tool for calculating the elastic-plastic response of structures subjected to an arbitrary dynamic loading function. As discussed in Chapter 3, WAC solves numerically the equation of motion for displacement using the average acceleration method described by Biggs (1964). The selected seal designs were subjected to the computed, normally reflected pressure-time curve...
shown in Figure 111. This analysis represents the worst case to which a seal could be subjected.

The selected cases and their responses to the worst case normally reflected pressure-time curve are shown in Table 30. The greatest displacement response was seen in the 6-ft by 20-ft seal, with a maximum deflection of 2.69 in. and a maximum support rotation of 4.27 deg. None of the analyzed seals reached the failure criterion of 6 deg of support rotation where complete loss of structural integrity is likely (as discussed earlier with Figure 5). In many cases, the seals reached more than 2 deg of support rotation where concrete crushing is likely and compression loads are carried by the steel reinforcement; however, the seals are still viable as protective structures. The deflection-versus-time curve from a WAC analysis for the 6-ft by 20-ft case is shown in Figure 116.

More than 2 deg of support rotation results in “moderate damage” and corresponds to a “low level of protection” according to Table 2, and B-1 of the report “Single Degree of Freedom Structural Response Limits for Antiterrorism Design” from the. Moderate damage is defined as, “Component has some permanent deflection. It is generally repairable, if necessary, although replacement may be more economical and aesthetic.” A low level of protection for a building is defined as “Unrepairable damage – progressive collapse will not occur. Space in and around damaged area is unusable.” At a low level of protection according to Table 2-2 (US Army Corps of Engineers – Protective Design Center 2006), “the majority of personnel in damaged area suffer minor to moderate injuries with the potential for a few serious injuries, but fatalities are unlikely.” A low level of protection is acceptable for the antiterrorism design of Department of Defense facilities, as specified in Table B-1 of the report “DOD Minimum Antiterrorism Standards for Buildings” (UFC 4-010-01; DOD 2007).

Table 30. Calculated response of various 120-psi seals subjected to the normally reflected pressure-time curve shown in Figure 111.

<table>
<thead>
<tr>
<th>Opening Size (ft)</th>
<th>Seal Thickness (in.)</th>
<th>Elastic Support Rotation Limit, $\Theta_e$ (deg)</th>
<th>Elastic Deflection Limit, $X_e$ (in.)</th>
<th>Ultimate Resistance, $r_u$ (psi)</th>
<th>Maximum Deflection from WAC, $X_{max}$ (in.)</th>
<th>Maximum Support Rotation, $\Theta_{max}$ (deg)</th>
<th>Building Level of Protection</th>
</tr>
</thead>
<tbody>
<tr>
<td>4x16</td>
<td>28.128</td>
<td>0.013</td>
<td>0.006</td>
<td>559.67</td>
<td>0.223</td>
<td>0.53</td>
<td>Medium</td>
</tr>
<tr>
<td>6x20</td>
<td>28.128</td>
<td>0.028</td>
<td>0.018</td>
<td>350.02</td>
<td>2.69</td>
<td>4.27</td>
<td>Low</td>
</tr>
<tr>
<td>7x20</td>
<td>40.128</td>
<td>0.017</td>
<td>0.013</td>
<td>397.96</td>
<td>1.10</td>
<td>1.50</td>
<td>Medium</td>
</tr>
<tr>
<td>10x16</td>
<td>52.128</td>
<td>0.019</td>
<td>0.020</td>
<td>333.40</td>
<td>1.97</td>
<td>1.88</td>
<td>Medium</td>
</tr>
<tr>
<td>12x20</td>
<td>52.128</td>
<td>0.029</td>
<td>0.037</td>
<td>293.45</td>
<td>3.40</td>
<td>2.70</td>
<td>Low</td>
</tr>
</tbody>
</table>
All of the designs evaluated in Table 30 exceed their elastic limit when subjected to the “worst case” normally reflected pressure-time history shown in Figure 111, and therefore would survive one application of the worst case pressure-time curve. To evaluate whether the designs could survive multiple applications of this worst case pressure-time curve, an artificial pressure-time curve was constructed by adding another worst case pressure-time curve to the original curve after an 80-msec delay, as shown in Figure 117. This artificial pressure-time curve was then used in WAC to determine the displacement response of the 6-ft by 20-ft seal subjected to repeated detonation waves. Figure 117 shows that, after the
arrival of the first detonation wave, the displacement is about 2.7 in., which is identical to that shown in Figure 116 and Table 30. However, after arrival of the second detonation wave, the displacement of the seal increases to more than 9 in. Thus, the 6-ft by 20-ft seal was not able to survive two detonations before exceeding 6 deg of support rotation. The 6-ft-by-20-ft seal had the largest displacement when subjected to one detonation wave, and it is possible that some of the other designs shown in Table 30 might survive at least two detonation waves, but this possibility was not considered further.

Figure 117. Calculated deflection-versus-time curve for a seal subjected to repeated “worst case”, normally reflected pressure waves.
8.5 Calculated response of 120-psi seals to a methane-air detonation pressure using UFC-3-340-02 (DOD 2008) response charts

UFC 3-340-02 (DOD 2008) provides response charts that can be used to calculate the dynamic response of a structure without the use of computational methods such as WAC. The dynamic response can be determined based on simplified load shapes and known parameters that describe the load and the resisting structure. The response charts were created by completing numerical integrations for many different cases, and then plotting those responses. These charts originate from Army technical reports and tri-service manuals that were developed before the proliferation of computers; however, the charts are still useful in the absence of SDOF software or the technical knowledge to conduct an SDOF analysis from fundamentals. An analysis using the response charts provides useful insights into the critical parameters that contribute to the response of a structural element subjected to a dynamic loading. The analysis considers the same 6-ft by 20-ft seal analyzed previously using the SDOF method, and follows the method presented in UFC 3-340-02 (DOD 2008) Section 3-19.3.4.

The shape of most pressure-time curves from explosions can be approximated with bi-linear triangular loads. On the basis of this assumption, response charts were developed for UFC 3-340-02. The number of response charts needed to characterize all of the useful loading cases is not too extreme.

The calculated displacement from an analysis using response charts will not match those from an SDOF analysis because of errors introduced when idealizing the pressure-time curve. As shown in Figure 118, two idealized triangular shapes are used to approximate the actual pressure-time curve. The area of overlap of these two ideal triangular loads results in an artificial increase in the associated impulse such that the impulse of the idealized load will be about 10% greater than the impulse from the actual load (Krauthammer 2008). The additional impulse from the overlapping region will cause the displacements determined by the response charts to be greater than the displacements determined by the SDOF analysis.

The bi-linear idealization of the worst-case, normally reflected pressure-time curve from Figure 111 is shown in Figure 118. The legs of the first triangle have lengths of \( P = 650 \text{ psi} \) and \( T = 16 \text{ msec} \). The legs of the second triangle have lengths of \( C_1P = 380 \text{ psi} \) and \( C_2T = 48 \text{ msec} \). Knowing \( P \) and \( T \),
the constants $C_1$ and $C_2$ are calculated as $C_1 = 0.58$ and $C_2 = 3$. The ultimate
elastic resistance, $r_u$, for this structure can be found analytically using the
calculation method presented in Section 7-2-1 of this report and
Equations 110 and 111. The elastic deflection of the seal at ultimate
resistance, $X_e$, is found with Equation 112, and the natural period of
vibration, $T_N$, is found with Equation 122. $r_u$ is determined to be 350 psi; $X_e$
is 0.018 in., and $T_N$ is 3.18 msec. The parameter $P/r_u$ is calculated as 1.86,
and the parameter $T/T_N$ is 5.03. All the values and parameters for the dis-
placement analysis using response charts are summarized in Table 31.
Table 31. Response chart parameters for a 6-ft by 20-ft mine seal.

<table>
<thead>
<tr>
<th>Opening Size (ft)</th>
<th>P (psi)</th>
<th>T (ms)</th>
<th>C₁P (psi)</th>
<th>C₂T (ms)</th>
<th>C₁</th>
<th>C₂</th>
<th>r₀ (psi)</th>
<th>X₀ (in)</th>
<th>Tₙ (ms)</th>
<th>P/r₀</th>
<th>T/Tₙ</th>
</tr>
</thead>
<tbody>
<tr>
<td>10x16</td>
<td>650</td>
<td>16</td>
<td>380</td>
<td>48</td>
<td>0.58</td>
<td>3.00</td>
<td>350</td>
<td>0.018</td>
<td>3.18</td>
<td>1.86</td>
<td>5.03</td>
</tr>
</tbody>
</table>

The parameters, \( P/r₀ \) and \( T/Tₙ \), can then be used in the response charts contained in UFC 3-340-02.

Based on the parameters \( C₁ \) and \( C₂ \), the correct response chart must be found in Chapter 3 of UFC 3-340-02. If a response chart with the exact values of \( C₁ \) and \( C₂ \) does not exist, linear interpolation may be used with the nearest response charts to determine the actual response. The nearest response charts for this application are for \( C₁ = 0.6810 \) and \( C₂ = 3.000 \) (Figure 119) and for \( C₁ = 0.4640 \) and \( C₂ = 3.000 \) (Figure 120).

The maximum dynamic displacement is found from Figures 119 and 120 given the parameters \( P/r₀ \) and \( T/Tₙ \). Following the green line in Figure 119, the maximum dynamic displacement is about 200 times the elastic deflection, or \( 0.018 \times 200 = 3.6 \text{ in} \). Similarly from Figure 120, the maximum dynamic displacement is about 125 times the elastic deflection or \( 0.018 \times 125 = 2.25 \text{ in} \).

Based on a linear interpolation between the values obtained from Figures 119 and 120, for a \( C₁ \) value of 0.58 the maximum dynamic displacement is about 167 times the elastic deflection, or \( 0.018 \times 167 = 3.01 \text{ in} \). A deflection of 3.01 in., as determined from the response charts, is about 10% greater than the maximum deflection of 2.69 in. obtained by the SDOF analysis.

The majority of the difference is caused by damping incorporated into the WAC program. Response charts in UFC 3-340-02 do not include damping because it is difficult to quantify, and the assumption that it does not exist is more conservative. WAC uses a damping coefficient equal to one half of a percent (0.5%) of critical damping.

The maximum deflections determined from the response charts reasonably match the results of the SDOF analysis. More importantly, both responses are less than structural failure at 6 deg of support rotation.
Figure 119. Maximum response of elastoplastic, single degree of freedom system bilinear triangular pulse for $C_1 = 0.6810$ and $C_2 = 3.000$ (from UFC 3-340-02, DOD 2008).

Figure 120. Maximum response of elastoplastic, single degree of freedom system bilinear triangular pulse for $C_1 = 0.4640$ and $C_2 = 3.000$ (UFC 3-340-02, 2008).
8.6 Response of a rock-bolt anchor foundation to a methane-air detonation pressure as calculated using WAC

The previous analysis showed that the seal structure did not fail when subjected to the “worst-case,” normally reflected pressure-time history curve shown in Figure 111. However, the seal will not remain in place without adequate supports. The rock-bolt anchor foundation was analyzed in the same manner as the mine seal. This analysis required a shear resistance-versus-deflection curve for the rock-bolt anchors. The following empirical load-slip relationship (Hawkins 1973) was used to approximate the rock-bolt anchor’s resistance-deflection curve.

\[
q = 2.86 \cdot K \cdot f_s' \cdot \sqrt{\frac{f_c'}{d} \cdot \log(240 \cdot S + 1) \times 10^{-3}} \quad (140)
\]

where

- \( q \) = shear stress (psi)
- \( K \) = 1.0 for dense aggregate, 0.85 for lightweight aggregate
- \( f_s' \) = ultimate tensile strength of steel (psi)
- \( f_c' \) = compressive strength of concrete (psi)
- \( d \) = steel diameter (in.)
- \( S \) = slip (in.)

Knowing the cross-sectional area of the rock bolts, the dimensions of the entry, and the number of rock-bolt anchors, the load-slip relationship can be expressed in terms of a pressure that is resisted by the rock-bolt anchors, using the following equation.

\[
p = \frac{q \cdot A_h \cdot N_{RB}}{H \cdot L} \quad (141)
\]

Substituting Equation 140 into Equation 141 gives a shear resistance versus deflection curve suitable for a SDOF analysis.

\[
p = 2.86 \cdot A_h \cdot K \cdot f_s' \cdot N_{RB} \cdot \frac{f_c'}{H \cdot L} \sqrt{\frac{f_c'}{d} \cdot \log(240 \cdot S + 1) \times 10^{-3}} \quad (142)
\]
where

\[ p = \text{total pressure resisted by rock bolts (psi)} \]
\[ A_b = \text{rock-bolt anchor cross-sectional area (in.}^2) \]
\[ N_{RB} = \text{number of rock bolts anchoring mine seal} \]
\[ H = \text{entry height (in.)} \]
\[ L = \text{entry width (in.)} \]

Hawkins (1973) recommends the use of 0.15 in. as the ultimate slip, which is conservative, since the average ultimate slip in Hawkins’ data was 0.159 in. To plot the resistance-versus-deflection function (Equation 142), the slip, \( S \), was varied incrementally in 15 steps of 0.01 in. each up to the ultimate slip of 0.15 in. The resistance-versus-deflection curve for the 6-ft-by-20-ft mine seal rock-bolt anchors is shown in Figure 121.

**Figure 121.** Resistance function of the rock-bolt anchors for a 6-ft-by-20-ft seal.
The load that the rock-bolt anchors must resist is the same load that must be transferred from the reinforced concrete seal structure to the surrounding rock. This load is the same as the resistance-versus-time output from a WAC analysis of the structure. Table 32 summarizes the results of the WAC analyses for the range of seal geometries considered previously.

Table 32. Calculated response of rock-bolt anchors subjected to the worst case normally reflected pressure-time curve shown in Figure 111.

<table>
<thead>
<tr>
<th>Opening Dimensions (ft)</th>
<th>Seal Thickness (in.)</th>
<th>Number of Rock Bolts, N_{RB}</th>
<th>Maximum Allowable Displacement (in.)</th>
<th>Ultimate Resistance, r_u (psi)</th>
<th>Maximum Response (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 × 16</td>
<td>28.128</td>
<td>120</td>
<td>0.15</td>
<td>771</td>
<td>0.134</td>
</tr>
<tr>
<td>6 × 20</td>
<td>28.128</td>
<td>156</td>
<td>0.15</td>
<td>535</td>
<td>0.139</td>
</tr>
<tr>
<td>7 × 20</td>
<td>40.128</td>
<td>216</td>
<td>0.15</td>
<td>634</td>
<td>0.107</td>
</tr>
<tr>
<td>10 × 16</td>
<td>52.128</td>
<td>260</td>
<td>0.15</td>
<td>668</td>
<td>0.136</td>
</tr>
<tr>
<td>12 × 20</td>
<td>52.128</td>
<td>320</td>
<td>0.15</td>
<td>548</td>
<td>Breakage</td>
</tr>
</tbody>
</table>

In one of the analyzed cases, the worst case pressure-time curve (Figure 111) leads to a rock-bolt anchor displacement beyond the ultimate allowable slip of 0.15 in. The rock-bolt anchors survive in the other four cases. An interesting result from analyzing the rock-bolt anchors against the resistance output of the mine seal is that increasing the resistance of the mine seal increases the load on the rock bolts. When the reinforced concrete seal is weak, damage is done to the seal, and the rock bolts see less load. When a mine seal has a higher ultimate resistance, less damage is done to the seal, and more load is transferred to the rock-bolt anchors.

The behavior of the seal after failure of the rock-bolt anchors is uncertain. The seal structure itself remains intact. Because the seal structure does not break up when the supports are compromised, debris will not be launched into the occupied area. Due to the geometry and mass of the seal, it seems unlikely that the seal will be displaced a significant distance down the entry, but this is not known. Failure of the rock-bolt anchors is actually a technical term meaning that their ultimate resistance has been exceeded. However, the intact body of a displaced seal may still succeed in protecting those in the active workings of the mine. It is important to note that this analysis does not include any interface resistance. It is assumed that all lateral force is resisted by the rock-bolt anchors. Since site-specific conditions will vary from mine to mine and place, there is no reliable frictional resistance that might supplement the rock-bolt anchor resistance. Unlike the flexural response of the seal itself, the plasticity of the rock bolts is
relatively limited and does not allow energy to be absorbed through large displacements. Ductile modes of failure are always preferred over brittle failures particularly when designing to resist catastrophic events such as blast pressures. Computational modeling has been conducted at the ERDC on rock-bolt anchoring techniques to improve rock-bolt ductility. The proposed design would take advantage of s-curve bending behavior by artificially installing conical air gaps around an initial length of the rock bolt in the concrete of the seal. This gap would allow the rock bolt to develop a diagonal portion without first crushing the concrete around it. Compared to the Hawkins displacements, the ultimate slip can be increased by more than 10 times by using this system. However, further work is required on this design before its usefulness and the accuracy of the model can be confirmed.

In some cases, the seal support strength needs to be improved to provide enough resistance to realistic loading conditions. The simplest way to increase the support strength is to increase the number of rock bolts that anchor the mine seal to the surrounding tunnel. However, as mentioned previously, if the ultimate resistance of the seal is decreased, then the load transferred to the rock-bolt anchors decreases. With design and analysis on a case by case basis, a balance could be struck between damage to the reinforced concrete seal and damage to the rock bolts. Alternative design concepts could also be used to increase the strength of the mine seal support.

### 8.7 Summary

An application of the “worst case”, normally reflected pressure-time curve from a methane-air detonation results in some plastic deformation in the reinforced concrete mine seal designs presented in Chapter 7. These designs meet the “elasticity of design” requirement in the Final Rule (2008) when subjected to the 120-psi pressure-time curve with an instantaneous rise time. Common practice for extreme loads, such as seismic or blast loading, typically allows deformation and resistance in the plastic range. In the worst case analysis, the support rotation induced by the worst case load was 4.27 deg. Failure criteria in UFC 3-340-02 (DOD 2008) say that, although the concrete begins to crush at 2 deg of support rotation, structural integrity remains until 6 deg of support rotation is reached. None of the reinforced concrete seal designs presented in Chapter 7 exceeded the 6 deg of support rotation criterion until the seal was impacted by a second, independent
detonation wave. However, the rock-bolt anchors that tie the seal structure to the entry did, in some cases, fail under this worst case load.

The displacement of the seal after failure of the rock-bolt anchors is uncertain, but it may still adequately protect the active workings. The simplest way to increase the anchor support capacity is to increase the number of rock-bolt anchors. Additional design and analysis may be able to create seals that balance suitable levels of damage to the reinforced concrete seal and the rock-bolt anchor supports. The reinforced concrete seal designs presented in Chapter 7 satisfy the Final Rule (2008) design load. However, when the seal is subjected to a worst case design load, a more robust design may be needed.
9 Protecting Seals from Blast Loads

9.1 Blast-wave attenuators

A blast-wave attenuator is “a stationary device used to reduce or lessen the blast wave effects by reducing the interior increase in pressure” (ASCE 1997). They are used to protect buildings in petrochemical facilities or underground rooms in military complexes from the effects of a nearby explosion. As an example of blast-wave attenuator construction, sand and rocks may be placed in the ventilation shafts of an underground bunker in order to decrease blast-wave effects while allowing ventilation airflow. Alternatively, a system of air intake baffles may be used that have little effect on airflow but that dramatically reduce the transmission of a short-duration blast pressure into the structure (Baker and Harrell 1992).

Blast attenuators are used to limit the effects of short blast durations (ASCE 1997), such as the detonation wave shown in Figure 109 that reaches 617 psi for few milliseconds, but rapidly decreases to less than 100 psi after less than 10 msec. As shown by Lusk et al. (2009) and Sapko et al. (2010), blast-wave attenuators can significantly reduce the peak explosion pressure on a coal mine seal and may reduce the seal’s structural requirements with a potential savings in construction costs.

At least three concepts have been developed for blast-wave attenuators in underground coal mines. Each of these concepts were studied experimentally, and a limited amount of test data were collected on gob exposed to blast waves to document its performance as a complete coal mine seal or as a seal component.

Lusk et al. (2009) described the gob plug seal concept (Figure 122) in which the coal mine roof is drilled and blasted into the entry to form the gob plug seal. Next, a seal with lower structural requirements is constructed in the entry behind the gob plug seal and, finally, the space between the seal and the blasted roof rock is filled with an inert material.

Sapko et al. (2010) presented the “shot rock” concept (Figure 123) in which the coal mine roof rock is also drilled and blasted into the coal mine entry to form a seal. Prior to blasting the roof rock, the entry may be partially filled with gob material obtained from other nearby underground sources. In this...
concept along with the prior “gob plug seal,” the height of the shot rock cavity and depth of the blast holes must be about 2.5 to 3.5 times the entry height in order to fill the entry and roof cavity with rock rubble.

Figure 122. Gob plug seal concept by Lusk, Unrug, and Perry (2009) composed of (1) blasted roof rock, (2) a light-weight construction seal, and (3) filler material between the seal and blasted rock.

Sapko et al. (2010) also presented the “fully stowed” concept (Figure 124) in which gob material obtained from within the mine is stowed in the entry along with wire mesh screen that has been attached to the roof, ribs and floor rock. The wire mesh anchors the gob material in place during an explosion event.

Figure 123. “Shot-rock” concept by Sapko et al. (2010) composed of (1) blasted roof rock and (2) an optional borrowed gob foundation.
9.2 Effectiveness of blast-wave attenuators – demonstration experiments

Lusk et al. (2009) conducted experiments to measure the blast-wave attenuation capabilities of blasted rock. Their tests measured pressures resulting from the detonation of a small, high-explosive charge inside an 8-ft-by-8-ft shock tube. Tests were conducted both with and without a blast-wave attenuator. The small-scale blast-wave attenuator was constructed from a pile of 1-in. crushed limestone rock that measured 5.5 ft long at its base and had an average thickness of about 3 ft. Without the attenuator, the high-explosive charge produced a blast wave beyond the planned seal location with magnitude of 27.16 psi and a nearly instantaneous rise time. With the attenuator, the measured pressure at the same location downstream of the crushed rock seal was 4.98 psi. The rise time of this blast wave was also increased and was estimated to be about 1 msec. This experiment demonstrated that a 3-ft-thick pile of crushed rock in an 8-ft-high tunnel can decrease the magnitude of a short duration blast wave by a factor of five or more.

Sapko et al. (2010) also obtained measurements of the blast-wave attenuation capabilities of blasted rock. Three full-scale tests of attenuators were conducted at the NIOSH Lake Lynn Laboratory (LLL) in an entry measuring about 20 ft wide by 7 ft high and using a methane-air explosion to generate the applied blast wave. The base of each test attenuator was blasted, run-of-mine limestone rock less than 18-in. diameter topped with crushed limestone less than 6-in. diameter.
Table 33 summarizes the dimensions of the three test attenuators and the pressure measurements upstream and downstream of the attenuators. In Tests 1 and 3 with no gap below the roof, the attenuator reduced the downstream static pressure by a factor of 67, or a reduction of 98.5%. This reduction is consistent in magnitude with experiments by Baker and Harrell (1992), who examined the effectiveness of baffles in suppressing air shocks entering intake airways of hardened structures. In Test 2, there was a 2.5-ft gap at the roof. Since the height of the entry was about 7 ft, the attenuator only reduced the cross-sectional area of the entry by about 64%. However, this reduction in cross-sectional area reduced the downstream static pressure by a factor of about 2.4. These experiments demonstrate that a simple rock pile placed from floor to roof and at least 10 ft long at its base can reduce the magnitude of short duration blast waves by a factor of 67.

Table 33. Summary of blast-wave attenuator tests at NIOSH-LLL.

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Attenuator Thickness</th>
<th>Upstream Static (side-on) Pressure (psi)</th>
<th>Downstream Static (side-on) Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>42 ft at base tapering to 15 ft at roof No gap at roof</td>
<td>47</td>
<td>0.7</td>
</tr>
<tr>
<td>2</td>
<td>48 ft at base tapering to 32 ft at top 2.5-ft gap at roof</td>
<td>50</td>
<td>21</td>
</tr>
<tr>
<td>3</td>
<td>42 ft at base tapering to 5 ft at roof No gap at roof</td>
<td>57</td>
<td>0.85</td>
</tr>
</tbody>
</table>

9.3 Effectiveness of blast-wave attenuators – numerical simulations

The experiments described above used either a small, high-explosive charge or a methane-air explosion to generate a blast wave. However, these test explosions do not capture the long-duration, constant-volume explosion pressure that can be expected from an actual methane-air explosion within a sealed area in an underground coal mine. Therefore, in the Sapko et al. (2010) study, J. R. Britt used a numerical gas explosion program called SHAMRC to calculate the pressures upstream and downstream of an attenuator resulting from a large methane-air explosion in a mine.
Figure 125 shows the mine layout containing the simulated explosion. This layout has an overall length of about 600 ft, which is the same as that at the NIOSH Lake Lynn Experimental Mine. The three drifts measure about 20 ft wide by 7 ft high and are connected by cross cuts about every 100 ft. In this model, three seals were placed at one end of the mine across the A, B, and C drifts, and three mine blast attenuators were placed about 100 ft in front of the seals. Figure 126 shows a cross section of the model mine blast attenuators. The lower portion of the attenuators is composed of well-packed gob material with little void space, whereas the upper part is made of large blocks with interconnected void spaces of about 20%. The entire mine model was filled with a homogeneous mixture of 9.5% methane in air, and then ignited at the end opposite the attenuators, as shown in Figure 9-4.

Figure 125. Layout of the Lake Lynn Experimental Mine used in a SHAMRC calculation of the explosion pressure upstream and downstream of simulated mine blast attenuators (Sapko et al. 2010).

Figure 126. Model attenuator used in the SHAMRC calculations. The lower portion is well-packed gob material with little void space and the upper portion is large blocks with interconnected void space of 20% (Sapko et al. 2010).
Figure 127 shows the calculated pressures upstream and downstream of the model attenuator directly aligned with the blast source in C-drift. The upstream static (side-on) pressure on the attenuator is about 240 psi, which produced a reflected pressure on the attenuator of almost 700 psi. The reflected pressure on the seal located downstream of the C-drift attenuator was initially about 20 psi. The reduction in pressure magnitude from upstream to downstream of the attenuator was a factor of about 35, which is consistent with the experimental results. However, as shown in Figure 127, the pressure on the seal downstream of the attenuator increases steadily to about 60 psi after 2 sec, and eventually reached the constant-volume explosion pressure of about 120 psi for a stoichiometric methane-air mixture.

9.4 Summary

Blast-wave attenuators are a well-developed and proven technique for decreasing the effects of short-duration blast waves. They are used in other civil industries and throughout the defense community to protect structures from short-duration blast waves. Both experiments and numerical simulations show that a blast-wave attenuator can decrease the magnitude
of a short-duration blast wave by a factor of 35 to 67 or more, depending on the characteristics of the attenuator, such as its density, porosity, permeability, and thickness. However, as the calculations shown in Figure 127 demonstrate, blast-wave attenuators will not necessarily decrease a much lower, long-duration blast pressure, such as the constant-volume explosion pressure resulting from a methane-air explosion within a sealed area.

Blast-wave attenuators can increase the rise time of short-duration blast waves and thereby decrease the rate of pressure change. This increase in rise time can have a positive effect on seal design by decreasing the dynamic load factor (DLF) from 2 to 1. This decrease effectively cuts the structural requirements for a seal in half. In the case of a concrete plug seal, the required design thickness could be halved.

The “Compliance Guide Questions and Answers” (MSHA 2008a) does not allow the use of blast-wave attenuators as a means to change the design pressure-time curves presented in the Final Rule (2008). However, this ruling should not prohibit the use of gob material in the construction of coal mine seals. Future coal mine seal designs could make effective use of this readily available material for seal construction.
10 Summary

10.1 Issues in coal mine seal design

The significant accomplishment of this report is the development of a design procedure for coal mine seals that:

1. Arises from the experience of the Department of Defense protective structure design community.
2. Follows recognized design codes and design criteria to ensure effective seal performance in coal mines.
3. Utilizes construction materials with known and well-understood properties.

Three characteristics of coal mine seal design make it challenging and unique, i.e., the pressure-time curves specified for coal mine seal design in the Final Rule (2008), the “elasticity of design” requirement, and the seal foundation. The Final Rule (2008) on sealing of abandoned areas describes four different pressure-time curves for seal design that depend on the seal application, i.e., either a mainline seal or a gob isolation seal, and whether the sealed area is monitored or not. These design pressure-time curves are similar to those of typical blast loads because of their short or instantaneous rise time for the pressure, but they also differ because of the long duration of the blast load. The impulse load produced by these long pressure durations present a significant challenge for seal designers.

The Final Rule (2008) specifies “elasticity of design,” which is understood to mean that the seal design must remain elastic when subjected to the design pressure-time curve. Protective structure design for military applications usually allows varying degrees of inelastic response, depending on the level of protection desired. In contrast, coal mine seals must withstand repeated loadings with the design pressure-time curve and still remain elastic in order to prevent noxious gases produced by an explosion within a sealed area from entering the active mine. The elasticity of design and the multiple loading requirements also present a significant challenge for seal designers.

Design of the seal foundation is the third challenge. Ground conditions can vary widely and may change over the life of the seal. The approach taken in
this study is to minimize the reliance on unknown or highly variable ground conditions for seal foundation design and to use alternative anchorage methods that are less dependent on these conditions. This study concludes that the best alternative is the use of rock-bolt anchors as the primary method to attach the seal to the surrounding rock.

10.2 Protective structure design and analysis methods

Protective structure design principles have been developed by the military for the design of facilities to resist the effects of explosions. These principles use known, well-understood materials, such as reinforced concrete, to design structures that will not fail suddenly in a catastrophic brittle mode but can fail gradually in a ductile mode. These principles are described and applied in many design manuals, most notably “Structures to Resist the Effects of Accidental Explosions” (Army TM 5-1300, Departments of the Army, Navy, and Air Force 1990) and “Design of blast resistant buildings in petrochemical facilities” (ASCE 1997). TM 5-1300 has been superseded by the Unified Facilities Criteria (UFC 3-340-02, DOD 2008), which is the major design manual applied in this report.

Three methods to analyze and design structures to resist dynamic loads are described. In the equivalent static method, the given dynamic load is transformed through the use of a dynamic load factor (DLF) into an equivalent static load that produces the same maximum displacement in the structure. The method is only applicable to elastic design. For the pressure-time curves with instantaneous rise times required by Final Rule (2008) for the design of coal mine seals, the DLF is always 2.0. For the other design pressure-time curves with 100- and 250-msec rise times, the DLF is always 1.0. The equivalent static method is the simplest and most widely used method for dynamic analysis and design.

Numerical solution of the equation of motion for an equivalent single-degree-of-freedom (SDOF) system is another method for the dynamic analysis of structures. The Wall Analysis Code (WAC; Slawson 1995) is a well-known example of an SDOF solution. This method considers the total mass of the structure and approximates the stiffness of the structure using a resistance function. Because of their simplicity, SDOF methods are also widely used for analysis and design of structures subjected to blast loads.

Other numerical methods, such as a dynamic finite element analysis, are also used for analysis and design of blast-loaded structures. These
multiple-degree-of-freedom methods consider both the mass distribution within a structure and the stiffness of the structure when calculating the internal dynamic stresses. Numerical methods produce the most accurate calculations of dynamic displacement, but they require reliable estimates of all material properties and boundary conditions.

10.3 Development of the wall analysis code (WAC)

This study makes extensive use of WAC-MS, derived from WAC (Slawson 1995), for analysis of and design of coal mine seals. WAC can calculate the dynamic displacement of a structure in the elastic range and into the plastic or inelastic range with good accuracy, as shown in Figure 81, which compares WAC-MS calculations to experimental test results. A 64-in.-high-by-34-in.-wide-by-4-in.-thick reinforced concrete test panel was subjected to a 50-psi blast load. In the experiment, the test panel displaced about 6.7 in. and underwent about 14 deg of rotation at the supports. The experiment loaded the test panel well beyond its elastic limit and far into the failure regime. A WAC-MS analysis of the test panel using the same blast load calculated a displacement of 5.6 in., which is about 16% less than experiment.

The resistance functions in WAC-MS are based on a flexural mechanism of wall response. However, many coal mine seals are thick compared to their width, and shear failure around the seal perimeter is the dominant mode of seal response. A shear resistance function was developed and implemented in WAC-MS.

A shear-stress resistance function for a 350-psi cement foam plug seal is presented in Figure 32. This function is based on full-scale tests conducted at the NIOSH Lake Lynn Laboratory. The ultimate shear-stress resistance was estimated to be 35.93 psi; however, the elastic shear-stress resistance was 84% of ultimate or 30.28 psi. For a 20-ft-wide-by-7-ft-high seal, the seal thickness required to resist the 120-psi instantaneous rise time pressure-time curve and remain elastic ranges from 7 ft to 25 ft, depending on the shear strength of the cement foam. With an elastic shear-stress resistance of 30 psi, a seal thickness of almost 21 ft is required in a 20-ft-wide-by-7-ft-high entry.

A similar shear-stress resistance function (Figure 45) is also presented for polyurethane foam and aggregate plug seals that is based on full-scale tests at Lake Lynn Laboratory. The ultimate shear-stress resistance was esti-
mated to be 21.4 psi, but the elastic shear-stress resistance was 19.0 psi. For a 20-ft-wide-by-7-ft-high seal, the seal thickness required to resist the 120-psi instantaneous rise time pressure-time curve is 29 ft, based on the ultimate shear-stress resistance.

10.4 Seal foundation analysis

For a seal structure to remain in place under a blast load, the seal foundation must provide sufficient anchorage capacity to resist the forces developed from the design pressure-time curve. In addition to calculating the dynamic response of a seal structure, WAC-MS also calculates the dynamic reactions that can be used to analyze and design a seal foundation.

For seal foundation analysis, a method is presented to analyze and calculate the resistance capacity of a hitch. The method is based on bearing analysis and requires a thorough assessment of the in-situ rock and coal properties, which are highly variable and difficult to characterize. Consequently, the use of a hitch to anchor a seal is not recommended.

The preferred method to anchor a seal is with rock bolts. The anchorage capacity of rock bolts is well understood and is less dependent on the surrounding rock strength. A rock-bolt anchor system can be designed with more certainty, and hence it is the recommended anchorage system. Again, WAC-MS can be used to calculate the reaction forces for a seal structure that are needed to design the anchorage system.

10.5 Seal structure analysis

Analyses are presented for three kinds of seals, i.e., reinforced concrete walls, unreinforced concrete plug seals, and a novel concept, the gob plug seal. The equivalent static design method was used to analyze a reinforced concrete wall subjected to the 120-psi pressure-time curve with an instantaneous rise time. The analysis treats the seal as a one-way wall that is simply supported at the ribs only. The final design for a 20-ft-wide-by-7-ft-high seal requires a 12-ft thickness with #10 reinforcing bars placed horizontally on 4-in. centers. The SDOF method was also applied in this study by using WAC to consider the seal as a two-way wall that is simply supported at the roof, floor, and ribs. The final seal design was reduced to a 3-ft thickness with #10 reinforcing bars placed horizontally on 4-in. centers.
The analysis of unreinforced concrete plug seals under the design load curve showed the distribution of compressive, tensile, and shear stresses in these thick structures. Tensile stresses in the concrete were negligible, and compressive stresses were less than 10% of the unconfined compressive strength of the concrete. However, the shear stresses around the perimeter of the seal were non-uniform and highest near the loaded seal face. This non-uniform shear-stress distribution in the seal leads to a non-uniform load distribution on the seal’s anchorage system that requires design consideration.

A preliminary analysis of a seal constructed of waste rock (gob) in an underground coal mine demonstrated the potential of this inexpensive, low-logistics method to construct a plug seal. The model gob plug seals (Figure 86) were 20 ft wide by 7 ft high and 18 ft thick with two thin concrete walls at the front and back ends of the gob pile to act as load collectors. The finite element method was used to calculate the seal’s displacement response when subjected to the 120-psi pressure-time curve with an instantaneous rise time. The behavior of the gob plug seal depends on the nature of the surrounding foundation rock. If it is weak, the seal fails by shear failure through the foundation as well as the seal; however, if it has sufficient strength, the gob plug seal appears to work well. Gob plug seals appear to be a viable option for coal mine seals, but not enough data exist to provide guidance for their design.

10.6 Design procedure for coal mine seals

A three-step design procedure was developed for general coal mine seals. The procedure consists of (1) design inputs, where the design pressure-time curve, material properties, and seal geometry are specified, (2) foundation design, where shear forces around the seal perimeter and the required seal anchorage are determined, and (3) seal structure design, where the seal thickness and internal seal reinforcement are determined. This process was applied to both reinforced concrete seal design and unreinforced concrete plug seal design. Design charts for these seal types were developed for the 120-psi pressure-time curves with instantaneous and 250-msec rise times. These design charts specify the number of rows of rock-bolt anchors required for the seal foundation and the thickness required for the seal structure. In the seal foundation design, rock-bolt anchors provide all the required shear resistance, and any friction with the surrounding roof, floor, and rib rocks is neglected, which is conservative. The design charts for reinforced concrete seals and unreinforced concrete
plug seals are consistent with many currently approved seal designs (MSHA 2013).

10.7 Additional considerations in coal mine seal design

As required in the Final Rule (2008), a seal design must resist the design pressure-time curve and address the “elasticity of design,” which requires that a seal’s response remain in the linear elastic range during loading (MSHA 2008a). The reinforced concrete seal designs presented in the design charts (Figure 103) meet the elasticity of design requirement for the 120-psi design curve with an instantaneous rise time. Based on measurements of the static (side-one) pressure-time curve from methane-air detonation experiments at NIOSH’s Lake Lynn Laboratory (Zipf et al. 2011), a reflected pressure-time curve was computed as shown in Figure 111. This computed curve has a peak pressure of about 2,275 psi with a duration of about 0.30 msec, and it decreases to the constant-volume explosion pressure of 120 psi after about 50 msec. This computed “worst case” pressure-time curve was used to calculate the displacement response of several typical seal designs from Figure 103. All of the considered seal structures survived this worst-case pressure-time curve. However, the computed displacements were no longer elastic, and plastic deformation occurred in all structures considered. The important point is that the seals survived. The computed displacements of the rock-bolt anchors in the seal foundations indicated failure in one of the cases considered.

Blast-wave attenuators are devices to reduce blast-wave effects that are commonly used in petrochemical and military facilities. Several concepts have been proposed, including the gob plug seal by Lusk et al. (2009) and the “shot rock” and “fully stowed” concepts by Sapko et al. (2010). Blast-wave attenuators can decrease the magnitude and increase the rise time of a short duration blast wave, which could reduce the required thickness of a coal mine seal by up to one-half. However, blast-wave attenuators will not necessarily eliminate the long-duration, constant-volume explosion pressure resulting from a confined methane-air explosion.
References


Sapko, M. J. 2013. Personal communication.


Appendix A: Wall Analysis Code for Mine Seals (WAC-MS)

Example: Reinforced concrete wall

This example takes the user step-by-step through the use of WAC-MS applied to reinforced concrete wall design. The following parameters were used to perform the analysis.

1. Entry geometry 7 ft by 20 ft
2. Final rule’s 120-psi instantaneous overpressure
3. Safety factor: 1.2
4. $F'_{c} = 5$ ksi

For these parameters, the required seal thickness from the design chart (Figure 103) is 40 in. The internal reinforcement is with 7/8-in.-diam (#7), Grade 60, steel bar, 6 in. on centers, both faces, and both directions, vertically and horizontally. Also required are four rows of rock-bolt anchors, 1.125-in.-diam (#9), Grade 60, steel bar for anchoring the seal to the surrounding rock.

When WAC-MS is first started, the main input window shown in Figure A1 appears. Subsequent screen captures in this discussion will illustrate problem definition and data entry into WAC-MS, followed by examples of analysis results that WAC-MS can provide.
Figure A1. Main input window for WAC-MS, which appears when starting the program. In this window, a reinforced concrete wall was selected. The user can give the project a title, select a unit system, define the pressure-time loading on the wall, specify the wall dimensions, specify the wall type (either reinforced concrete wall or plug seal), and define the section properties of the wall. When WAC-MS has all the requisite input parameters, the analysis can be run by clicking the “Run” button in the lower right.
Figure A2. Clicking on the “Define Load Case” button in the main input window (Figure A1) leads to this window where the user selects a pressure-time loading function. All the design loads from the Final Rule (2008) are included. The user can also define a unique blast load. Safety factor is defined on this screen.
Figure A3. Clicking on the “Define Section Properties” button in the main input window (Figure A1) leads to this window where the user defines the properties and geometry of the internal steel reinforcement. Strength of the concrete and reinforcement bar is defined here. The user can define both vertical and horizontal reinforcement. Both tensile and compressive reinforcement is specified. The distances are measured from the blast-loaded side of the wall. The defined wall is 40.128 in. thick (Figure A1). The reinforcement bar is centered 3.31 in. from each face, which is 3.31 in. from the outside or compression face and 40.13 - 3.31 = 36.82 in. from the inside or tension face.
Figure A4. Clicking on the “Select Support Condition” button in the define section properties window (Figure A3) leads to this window where support conditions are defined. In this example, all edges are simply supported, i.e., free to rotate or zero moment conditions. After clicking “OK”, the user returns to the main input window (Figure A1). The problem can be run by clicking the “Run” button in the main input window (Figure A1).

![Reinforced Wall Support Conditions](image)

Figure A5. After clicking the “Run” button in the main input window (Figure A1), WAC-MS solves for the response of the structure and displays this window, which shows the maximum flexural displacement (0.010667 in.) and the elastic limit (0.0147206 in.). Because the maximum displacement is less than the elastic limit, this structural design is still linear elastic, and it meets the “elasticity of design” requirement in the Final Rule (2008). Clicking the “OK” button takes the user to the main output window shown in Figure A6.
Figure A6. Main output window for WAC-MS program showing output text of key calculation results. This window is similar to the main input window (Figure A1). Key calculation results such as maximum flexural displacement are shown in the box on the right. Clicking the “Plot” button enables display of various plots of the calculated response.
Figure A7. Main output window for WAC-MS program showing output plot of displacement versus time. With the menu box in the upper left, the user can select various calculated results to display such as displacement, velocity, acceleration, or resistance versus time.
Figure A8. Main output window for WAC-MS program showing output plot of resistance versus displacement. Clicking on the "View Advanced Graph" button below the plot allows the user to examine any plot in greater detail.
Figure A9. With the View Advance Graph option, the user can zoom-in to closely examine various calculated results. The two black vertical lines indicate the maximum displacement for this design (about 0.011 in.) and the elastic limit for this design (about 0.015 in.).
Figure A10. Selecting “Horizontal Support Dynamic Reactions” from the menu box in the main output window then clicking the “View Advance Graph” button results in this plot of dynamic reaction along the horizontal supports (roof and floor). The rock-bolt anchors must resist a maximum reaction force of about 110,000 lb/ft along the 20-ft length of the roof and floor.
Figure A11. Selecting “Vertical Support Dynamic Reactions” from the menu box in the main output window then clicking the “View Advance Graph” button results in this plot of dynamic reaction along the vertical supports (left and right ribs). The rock-bolt anchors must resist a maximum reaction force of about 110,000 lb/ft along the 7-ft length of the left and right ribs.

Example: Analysis of cement foam plug seal

This example takes the user step-by-step through the use of WAC-MS applied to plug seal design. The following parameters were used to perform the analysis.

1. Entry geometry: 7 ft by 20 ft
2. Final rule’s 120-psi instantaneous overpressure
3. Safety factor: 1.0
4. $F_c' = 350$ psi
5. The lower- and upper-bound maximum shear strengths are 29.9 psi and 112.3 psi, respectively, for cement foam with compressive strength of 350 psi.
This WAC-MS analysis is identical to one presented earlier in Chapter 4 of this report. That analysis used the equivalent static method, and the required seal thickness was calculated as 298 in. with the lower bound estimate of shear strength and 80 in. with the upper bound estimate.

When WAC-MS is first started, the main input window shown in Figure A12 appears. Subsequent screen captures in this discussion will illustrate problem definition and data entry into WAC-MS, followed by examples of analysis results that WAC-MS can provide.
Figure A12. Main input window for WAC-MS that appears when starting the program. In this window, a plug seal was selected. The user can give the project a title, select a unit system, define the pressure-time loading on the wall, specify the wall dimensions, specify the wall type (either reinforced concrete wall or plug seal), and define the section properties of the wall. The “Define Load Case” button is used to select the 120-psi pressure-time curve with instantaneous rise time. For the lower- and upper-bound analyses, trial plug thicknesses of 296 and 79 in., respectively, are input. When WAC-MS has all the requisite input parameters, the analysis can be run by clicking the “Run” button in the lower right.
Figure A13. Clicking the “Define Shear Resistance Function” in the main input window leads to this window where the user defines a shear resistance function. By default, WAC-MS contains the generic shear resistance function presented in Chapter 4 Table 19 of this report. For the lower- and upper-bound analyses, $t_{\text{max}}$ is 29.9 and 112.3 psi, respectively. In this particular shear resistance function, the elastic limit occurs at point number 3 of the curve. Before this point, the resistance function is linear and resulting designs can satisfy the “elasticity of design” requirement in the Final Rule (2008).
Figure A14. After clicking the “Run” button in the main input window (Figure A12), WAC-MS solves for the response of the structure and displays this window, which shows the maximum shear displacement and the elastic limit. In both analyses, the maximum displacement is less than the elastic limit. The structural designs are still linear elastic, and they would meet the “elasticity of design” requirement in the Final Rule (2008). Clicking the “OK” button takes the user to the main output window shown in Figure A15.
Figure A15. Main output window for WAC-MS program showing output text of key calculation results. This window is similar to the main input window (Figure A12). Key calculation results such as maximum shear displacement are shown in the box on the right. Clicking the “Plot” button enables display of various plots of the calculated response.
Figure A16. Main output window for WAC-MS program showing output plot of resistance versus time. Clicking on the “View Advanced Graph” button below the plot allows the user to examine any plot in greater detail.
This report describes structural analysis and design methods applied to coal mine seals. Three characteristics of coal mine seal design make it challenging and unique, i.e., the pressure-time curves specified for coal mine seal design, the “elasticity of design” requirement, and the seal foundation. An important protective structure design principle to apply in coal mine seal design is that a design should not fail suddenly in a catastrophic brittle mode, but should fail gradually in a ductile mode.

A three-step design procedure for coal mine seals is presented that follows design codes and design criteria developed by the military for design of protective structures. The procedure involves (1) design inputs where the design pressure-time curve, material properties, and seal geometry are specified, (2) foundation design where shear forces around the seal perimeter and the required seal anchorage are determined, and (3) seal structure design where the seal thickness and internal seal reinforcement are determined. Design charts for these seal types are developed for the 830-kPa (120-psi) pressure-time curves with instantaneous rise time. The reinforced concrete designs in the chart are able to withstand a worst-case detonation wave; however, the designs are no longer elastic and permanent deformation occurs.

15. SUBJECT TERMS
Coal Mine Safety
Mine Seals
Structural Analysis
Blast Resistant Design
Protective Structures
Protective Design

16. SECURITY CLASSIFICATION OF:
19a. NAME OF RESPONSIBLE PERSON
Dr. Will McMahon

Form Approved
OMB No. 0704-0188

Please DO NOT RETURN YOUR FORM TO THE ABOVE ADDRESS.