MILITARY HANDBOOK

RIGID PAVEMENT DESIGN FOR AIRFIELDS

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NEW PAGE	DATE	SUPERSEDED PAGE	DATE
v	31 August 1992	v	30 April 1987
vi	31 August 1992	vi	30 April 1987
ix	31 August 1992	ix	30 April 1987
x	31 August 1992	x	30 April 1987
xi	31 August 1992	xi	30 April 1987
xii	31 August 1992	xii	30 April 1987
xiii	31 August 1992	xiii	30 April 1987
xiv	31 August 1992	xiv	30 April 1987
xv	31 August 1992	New Page	
1	31 August 1992	1	30 April 1987
2	31 August 1992	2	30 April 1987
31	31 August 1992	31	30 April 1987
32	31 August 1992	32	30 April 1987
33	31 August 1992	33	30 April 1987
34	31 August 1992	34	30 April 1987
35	31 August 1992	35	30 April 1987
36	31 August 1992	36	30 April 1987
37	31 August 1992	37	30 April 1987
38	31 August 1992	38	30 April 1987
39	31 August 1992	39	30 April 1987
40	31 August 1992	40	30 April 1987
43	31 August 1992	43	30 April 1987
44	31 August 1992	44	30 April 1987
53	31 August 1992	53	30 April 1987
54	31 August 1992	54	30 April 1987
54a	31 August 1992	New Page	
73	31 August 1992	73	30 April 1987
74	31 August 1992	74	30 April 1987

NEW PAGE	DATE	SUPERSEDED PAGE	DATE
79	31 August 1992	79	30 April 1987
80	31 August 1992	80	30 April 1987
87	31 August 1992	87	30 April 1987
88	31 August 1992	88	30 April 1987
107	31 August 1992	107	30 April 1987
108	31 August 1992	108	30 April 1987
119	31 August 1992	119	30 April 1987
120	31 August 1992	New Page	
121	31 August 1992	121	30 April 1987
122	31 August 1992	122	30 April 1987
123	31 August 1992	123	30 April 1987
124	31 August 1992	124	30 April 1987
125	31 August 1992	125	30 April 1987
126	31 August 1992	126	30 April 1987
127	31 August 1992	New Page	
128	31 August 1992	New Page	
129	31 August 1992	New Page	
130	31 August 1992	New Page	
131	31 August 1992	New Page	
132	31 August 1992	New Page	

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FOREWORD

This military handbook is one of a series developed from an evaluation of facilities in the shore establishment, from surveys of the availability of new materials and construction methods, and from selection of the best design practices of the Naval Facilities Engineering Command, other Government agencies, and the private sector. It uses, to the maximum extent feasible, national professional society, association, and institute standards in accordance with NAVFACENGCOM policy. Deviations from these criteria in the planning, engineering, design, and construction of naval shore facilities, cannot be made without prior approval of NAVFACENGCOM Headquarters (Code 04).

Design cannot remain static any more than can the naval functions it serves or the technologies it uses. Accordingly, recommendations for improvement are encouraged from within the Navy and from the private sector and should be furnished to Commander, Western Division, Naval Facilities Engineering Command (Code 406), P.O. Box 727, San Bruno, CA 94066.

THIS HANDBOOK SHALL NOT BE USED AS A REFERENCE DOCUMENT FOR PROCUREMENT OF FACILITIES CONSTRUCTION. IT IS TO BE USED IN THE PURCHASE OF FACILITIES ENGINEERING STUDIES AND DESIGN (FINAL PLANS, SPECIFICATIONS, AND COST ESTIMATES). DO NOT REFERENCE IT IN MILITARY OR FEDERAL SPECIFICATIONS OR OTHER PROCUREMENT DOCUMENTS.

AIRFIELD PAVEMENT CRITERIA MANUALS

Criteria Manual	Title	PA
MIL-HDBK-1021/1	Airfield Geometric Design	SOUTHDIV
MIL-HDBK-1021/2	General Concepts for Pavement Design	WESTDIV
DM-21.03	Flexible Pavement Design for Airfields	ARMY
MIL-HDBK-1021/4	Rigid Pavement Design for Airfields	WESTDIV
DM-21.06	Airfield and Subsurface Drainage Pavement Design for Frost Conditions	SOUTHDIV
DM-21.09	Skid-Resistant Runway Surfaces	HEADQUARTERS

Page

Page

Page

FIGURES

Page

FIGURES (Cont.)

TABLES

Page

TABLES

Section 1: INTRODUCTION

1.1 Scope. This handbook includes information and criteria for designing rigid airfield pavements for all Navy and Marine Corps airfield facilities. These criteria encompass subgrade soils, base course materials, stabilized layers, concrete materials, aircraft loadings, and rigid pavement thickness design. The rigid pavement thickness design is based on the Westergaard theory of a slab loaded at the interior resting on a dense liquid foundation. Criteria are also presented for strengthening existing rigid pavements.

1.2 Cancellation. This military handbook, MIL-HDBK-1021/4, Rigid Pavement Design For Airfields, cancels and supersedes Chapter 4 (Section 5) and Chapter 5 (Section 3) of NAVFAC DM-21, Airfield Pavements, June 1973.

1.3 Related Criteria. Additional criteria related to the design of rigid airfield pavements may be found in the following applicable publications:

Subject	Source
Architecture	
Noise criteria	DM-1.03
Civil Engineering	
Pavements	DM-5.04
Soils and Foundations	
Soil mechanics	DM-7.01
Foundations and earth structures	DM-7.02
Airfield Pavement Design	
Airfield pavement design, evaluation, and marking; soil stabilization for pavement; design for frost conditions and subsurface drainage; skid-resistant runway surfaces	MIL-HDBK-1021/1 MIL-HDBK-1021/2 DM-21.03 DM-21.06 DM-21.09 Army TM 5-822-4
Petroleum Fuel Facilities	
Direct fueling stations for fixed-wing aircraft and helicopters; fuel distribution; dispensing fuel to aircraft and surface vehicles; operating fuel storage	DM-22
Airfield Lighting	MIL-HDBK-1023/1

Navigational and Traffic Aids	NAVAIR 51 50-AAA-2
Maintenance Facilities	
Hangars; power check pad; boresight range; weapons alignment facility	MIL-HDBK-1028/1 MIL-HDBK-1028/6

Section 6: PORTLAND CEMENT CONCRETE MATERIALS

6.1 Introduction. This section provides guidelines for the selection of cement types, aggregates, and admixtures. Recommended aggregate and concrete testing procedures are also covered. See NFGS-02520, Portland Cement Concrete Pavement for Roads and Airfields, for additional information.

6.2 Heat Resistant Concrete for F/A-18 Aircraft. Pavement areas that are exposed to jet blast for an extended period of time may experience damage in the form of surface scaling. Surface scaling is a result of differential expansion and vaporization of moisture in the concrete. Aggregates having a low coefficient of thermal expansion and low porosity, such as basalt, will provide greater resistance to surface scaling than will carbonate aggregates. When designing parking apron pavements for the F/A-18 provide heat resistant concrete in each parking position to withstand the heat from the auxiliary power unit (APU). The aggregate shall be trap rock, fine grained, of the diabase or basalt variety, free of voids, quartz, and zeolites, all as described in ASTM C294. The aggregates shall meet the requirements of ASTM C33 and shall be tested for reactivity with alkalies in accordance with ASTM C289. Fine aggregate shall be produced from the same material as the coarse aggregate.

6.3 Cement Types. See ASTM C150 for Portland cement specifications. The five general ASTM designations for Portland cement are listed below.

6.3.1 Type I. Use in general construction where no special properties are needed and where concrete is not exposed to sulfate attack.

6.3.2 Type II. Use when concrete is exposed to moderate sulfate action. Type II generates less heat than Type I during hydration and may be used for mass projects or when placing concrete during hot weather.

6.3.3 Type III. Use when high early strength is required. It is used when forms must be removed quickly and when a reduced curing period is necessary before opening to traffic. Type III generates more heat than Type I during hydration and may be used when placing concrete in cold weather.

6.3.4 Type IV. Use when low heat of hydration is required. Strength development is slower than Type I.

6.3.5 Type V. Use when concrete is exposed to extreme sulfate action.

Types IA, IIA, and IIIA are Portland cements into which an air-entraining addition is blended during manufacture. Do not use Types IA, IIA, and IIIA because of the inability to control air content and lack of uniformity at the concrete plant.

6.3.6 Criteria for Selection of Cement Types. The primary factors in selecting a cement are resistance to detrimental chemical actions (e.g., sulfate attack), construction considerations (e.g., temperature at time of construction, rapid strength gain), and availability.

6.3.6.1 Chemical Actions. If the concrete will be exposed to sulfates, choose a sulfate-resistant cement. Table 8 describes sulfate concentrations and the recommended cement types.

6.3.6.2 Construction Considerations. The temperature at the time of construction may dictate the selection of the cement type. If construction is performed in cold weather, a Type III cement may be used to generate a greater amount of heat during hydration and accelerate hardening. If construction is performed in hot weather, a Type IV cement may be used to reduce the heat of hydration and slow hardening. If rapid strength gain is required in order to return a facility to service as soon as possible, a Type III cement may be used to provide rapid hardening and high early strength gain.

6.3.6.3 Availability. The availability of a cement type may dictate which cement type is chosen. Types I and III are generally readily available throughout the United States. Type II is common in many western states where the soils contain a greater sulfate concentration. Types IV and V may only be available in areas where a special market exists and they are routinely used.

Table 8
Types of Cement Required for Concrete
Exposed to Sulfate Attack

Relative Degree of Sulfate Attack	Percentage Water-Soluble Sulfate (as SO_4) in Soil Samples	Sulfate (as SO_4) in Water Samples (parts/million)	Cement Type
Negligible	0.00 to 0.10	0 to 150	I
Positive	0.10 to 0.20	150 to 1,500	II
Severe	0.20 to 2.00	1,500 to 10,000	V*
Very Severe	2.00 or more	10,000 or more	V+pozzolan**

Source: U.S. Bureau of Reclamation, Concrete Manual, 1979

* Or approved Portland-pozzolan cement providing comparable sulfate resistance when used in concrete.

** Should be approved pozzolan that has been determined by tests to improve sulfate resistance when used in concrete with Type V cement.

6.4 Concrete Aggregates. Aggregates have an important influence on the properties of Portland cement concrete. Aggregates are an economical filler and provide concrete with dimensional stability and wear resistance. The following sections provide information on aggregate types, recommended gradations, performance, and the use of recycled Portland cement concrete for use in a surface concrete mix.

6.4.1 Aggregate Types. Aggregates are classified as light weight, normal weight, or heavy weight. This discussion is limited to normal weight aggregates which are used in the majority of pavement construction.

Aggregates must be hard and strong, inert, and free from any impurities such as silt, clay, or organic matter. Soft aggregates can limit the strength and reduce durability of the concrete. Impurities may increase water requirements and interfere with the hydration reactions.

The aggregate shape and texture affect the workability of fresh concrete and the strength of the hardened concrete. Angular, elongated, or irregular shaped aggregates will increase paste requirements because of increased interparticle interaction. Elongated aggregates may also cause problems with segregation during handling. While rough, textured surfaces may increase mechanical bond, an elongated shape may also indicate an aggregate with weak fracture planes. This may have an adverse effect on the hardened concrete strength.

Consider the durability of the aggregate when designing a concrete mix. Both the chemical and physical durability of the aggregate must be considered. Physical durability concerns the soundness, wear resistance and freeze-thaw characteristics of the aggregate. See NFGS-02520 for specifications on soundness, wear resistance and freeze-thaw damage.

Chemical durability concerns the cement-aggregate reactivity. Reaction of alkali in cement with siliceous or carbonate aggregates causes expansion damage to the concrete, resulting in map cracking. Sedimentary rocks such as limestone, shale, and sandstone which contain amorphous silica such as opal and chert are susceptible to reactivity with alkalies in cement. The alkali-silica reaction can be controlled with a pozzolanic admixture. Some carbonate rocks such as dolomitic limestones can also react with alkali in cement and cause expansive damage to concrete. The alkali-carbonate reaction cannot be controlled with pozzolans. Specify cements low in alkali content for use with aggregates susceptible to alkali-carbonate reactivity. See ASTM C227 and ASTM C289 for specifications on alkali reactivity.

6.4.2 Gradation. Aggregate gradation influences the amount of cement that is required, the handling characteristics of the plastic concrete, and the hardened concrete properties. See ASTM C33 for gradation limits for coarse and fine aggregates.

6.4.3 Maximum Aggregate Size. For pavement slabs, the maximum aggregate size should not exceed 1.0 to 1.5 inches (25 to 38 mm). If reinforcement is used, the maximum size should not exceed three fourths of the minimum clearance between reinforcing bars or reinforcing bars and forms.

6.4.4 Performance of Local Aggregates. Use suitable local aggregates if available. This should result in lower costs for transporting materials. Thoroughly test aggregates for resistance to "D" cracking, reactivity, soundness, and abrasion resistance. Primary concerns with local aggregates center around "D" cracking and reactivity.

6.4.4.1 "D" Cracking. Durability or "D" cracking refers to deterioration along a joint or crack in a slab caused by the concrete's degradation due to freeze-thaw cycles. "D" cracking is related to aggregates that are expansive when saturated with water and then frozen. The pressure exerted by the repeated expansion is great enough to fracture the concrete and cause "D" cracking. "D" cracking usually begins at joints and cracks because free moisture is present and will appear as a pattern of cracks running parallel to the joint or linear crack. A dark coloring can usually be seen around fine durability cracks. Cracking may eventually lead to disintegration of the concrete within 1 to 2 feet (305 to 610 mm) of the joint or crack. Local aggregates must be tested for soundness and durability to ensure acceptable performance in areas where many cycles per year of freeze-thaw of the concrete slab occurs. Aggregates must satisfy ASTM C88 and ASTM C666. If aggregates are susceptible to "D" cracking the maximum aggregate size should be 1.0 inch (25 mm).

6.4.4.2 Reactive Aggregates. The most common chemical reactivity problem is the alkali-aggregate reaction. This reaction can take the form of an alkali-silica reaction or an alkali-carbonate reaction. The alkali-aggregate reaction results in the formation of a gel which is accompanied by a volume expansion. This volume expansion is sufficient to cause cracking throughout the concrete slab. The primary factors that control alkali-aggregate expansion are the presence of reactive silica, dolomite, or calcite; particle size of reactive material, amount of reactive alkali in cement, and amount of available moisture. Local aggregates must be evaluated for alkali-aggregate reactivity and satisfy ASTM C289.

6.4.5 Recycled Portland Cement Concrete. Recycled Portland cement concrete pavement can serve as a high-quality and low-cost aggregate in Portland cement concrete. The recycled concrete must be properly crushed and sized to meet aggregate gradation requirements.

Once the material has been properly sized, it must be tested for quality as outlined in this section. Recycled concrete aggregate meeting all quality requirements for new aggregate can be used in Portland cement concrete pavement. Lower-quality recycled concrete aggregate can be considered for use in base courses.

6.5 Admixtures. An admixture is a material other than water, aggregate, or cement that is added to the concrete batch immediately before or during mixing. Admixtures can be used to improve the handling and consolidation of plastic concrete or the performance and material characteristics of hardened concrete. Table 9 summarizes the principal advantages and disadvantages of the major types of admixtures. The major types of admixtures are described below. See NFGS-02520 for additional information and specifications.

Table 9
Admixture Effects on Concrete Properties

ADMIXTURE TYPE	FRESH CONCRETE	HARDENED CONCRETE
Air-Entraining	improved workability reduced bleeding reduced segregation	reduced strength improved freeze-thaw resistance improved sulfate resistance
Water-Reducing	improved workability may increase bleeding increase entrained air above level desired	increased strength increased impermeability improved durability
Set-Retarding	reduced workability increase entrained air above level desired	increased shrinkage reduced early strength
Set-Accelerating	-	increased early strength may decrease final strength increased shrinkage reduced sulfate resistance reduced alkali-aggregate resistance
Pozzolanic Material	improved workability	improved sulfate resistance increased impermeability improved durability increased strength

6.5.1 Air-Entraining Admixtures. Use air-entraining admixtures to improve workability and freeze-thaw durability and provide better overall resistance to sulfate actions. Air entrainment also reduces bleeding and segregation that may occur during transportation and handling. Entrained air is required in all Navy and Marine Corps concrete pavements. See ASTM C260 for specifications on air-entraining admixtures.

6.5.2 Water-Reducing Admixtures. Use water-reducing admixtures to lower the water required to attain a given slump. This generally leads to an increase in strength, impermeability, and durability. Possible disadvantages include an increase in bleeding and air entrainment. The active ingredient in water-reducing compounds is adsorbed at the solid-water interface. This neutralizes the surface charge on the particles so that all particle surfaces carry a like charge and repel each other. The particles are fully dispersed in the cement paste and free the water to reduce the viscosity of the cement paste. See NFGS-02520 and ASTM C494 for specifications on water-reducing admixtures.

6.5.3 Set-Retarding Admixtures. Use set-retarding admixtures to delay setting time when placing concrete in high temperatures, when delays may be unavoidable, or when placing large volumes of concrete. Set-retarding admixtures act to decrease the rate of early hydration. Possible detrimental effects on concrete properties include an increase in the rate of loss of workability even though the setting time is extended, a reduction in the early strength of the concrete, and an increase in shrinkage and creep. See NFGS-02520 and ASTM C494 for specifications on set-retarding admixtures.

6.5.4 Set-Accelerating Admixtures. Use set-accelerating admixtures to facilitate early strength gain and to overcome slow hydration rates due to low temperatures. The primary mechanism of accelerators is to increase the rate of the early stages of hydration. This results in increased 6-hour to 24-hour flexural strengths; however, strength gains diminish with time and the final flexural strength of the mix may be less than a concrete without the accelerating admixture. Accelerating admixtures may also increase shrinkage and creep. If calcium chloride is used, resistance to sulfate attack may be reduced and the alkali-aggregate reaction may be aggravated. See NFGS-02520 and ASTM D98 for specifications on set-accelerating admixtures.

6.5.5 Mineral Admixtures. Mineral admixtures can be subdivided into the following general categories.

6.5.5.1 Materials of Low Reactivity. These materials are used to increase the workability of plastic concrete which is deficient in fines. Common materials are ground limestone, dolomite, quartz and rock dust. Do not use these types of mineral admixtures because they seldom contribute to strength and durability of the hardened concrete.

6.5.5.2 Cementitious Materials. Use these materials to improve the strength of hardened concrete. Common materials are natural cements, hydrated lime, and blast furnace slag.

6.5.5.3 Pozzolanic Materials. Use these materials to improve the workability of plastic concrete, the impermeability and durability of hardened concrete. The addition of a pozzolan also improves the resistance to sulfate attack and alkali-silica reactivity. Common materials include natural pozzolans and synthetic pozzolans. Natural pozzolans such as chert and shale are seldom used because grinding to cement fineness is required. Synthetic pozzolans include fly ash and boiler slag. Fly ash is most commonly used because it is finely divided and can improve the workability of the plastic concrete without a large increase in water requirements.

The Navy and Marine Corps do not allow fly ash to be substituted for Portland cement; however, concrete designs using fly ash as a substitute for Portland cement will be evaluated on a project by project basis and must be approved by the Naval Facilities Engineering Command Field Division. Pozzolans should conform to ASTM C618. See NFGS-02520 for guide specifications on pozzolans.

6.5.6 Factors in Selecting Admixtures. The primary factor in deciding when to use an admixture is the effect the admixture will have on the plastic and hardened properties of the concrete. Primary concerns may include workability, placing characteristics, and early strength development. Incorporate admixtures during the mix design stage in order to evaluate the effect of admixtures.

6.6 Testing Procedures. See NFGS-02520 for guide specifications on field testing and sampling.

6.6.1 Aggregate Tests. The following sections provide brief information on the major specification tests.

6.6.1.1 Sieve Analysis and Gradation. Particle size distribution has a major effect on the water requirements and handling properties of fresh concrete. See ASTM C136 for specifications on sieve analysis and ASTM C33 for gradation limits on coarse and fine aggregates.

6.6.1.2 Specific Gravity and Absorption. Specific gravity and absorption are necessary for calculating batch quantities for mix design. Absorption is also used to evaluate freeze-thaw durability. See ASTM C127 and C128 for specific gravity and absorption of coarse aggregates and fine aggregates.

6.6.1.3 Unit Weight. The unit weight of aggregates is required for computing the volume of voids and approximating the amount of cement paste required in mix design. See ASTM C29 for specifications on unit weight.

6.6.1.4 Surface Moisture. The amount of surface moisture on the aggregate must be known to control mix design. ASTM C70 contains specifications for the measurement of surface moisture.

6.6.1.5 Soundness. Use soundness tests to evaluate the resistance to disintegration in aggregates due to cycles of wetting and drying or freezing and thawing. See ASTM C88 for specifications on the soundness tests. ASTM C666 provides specifications for freezing and thawing tests on aggregates.

6.6.1.6 Resistance to Abrasion. Abrasion tests are used to measure the deterioration or degradation of aggregates due to impact and surface abrasion. See ASTM C131 and C535 for specifications on abrasion tests.

6.6.1.7 Cleanliness and Deleterious Substances. Tests for deleterious substances and impurities are listed in Table 10.

Table 10
Tests for Deleterious Substances

TEST DESCRIPTION	ASTM DESIGNATION
Test for organic impurities in sands	C40
Effect of organic impurities on strength	C87
Test for lightweight pieces	C123
Test for clay lumps and friable materials	C142

6.6.2 Concrete Tests. Concrete tests and specifications are necessary to insure that hardened concrete with the required properties is attained. These tests can be broken down into tests for fresh concrete and tests for hardened concrete. The following sections provide some limited information on the major specification tests. See NFGS-02520 for additional information.

6.6.2.1 Fresh Concrete Tests. Concrete should be manufactured and transported to the job site in accordance with ASTM C94. Fresh concrete tests are summarized in Table 11.

Table 11
Fresh Concrete Tests

TEST DESCRIPTION	ASTM DESIGNATION
Making and curing laboratory concrete test specimens	C192
Making and curing field concrete test specimens	C31
Slump test	C143
Unit weight, yield, and air content (gravimetric)	C138
Air content of fresh concrete (volumetric method)	C173
Air content of fresh concrete (pressure method)	C231
Air content of fresh concrete (gravimetric method)	C138
Cement content	C138
Bleeding	C232
Time of setting by penetration	C403
Early volume changes	C827
Water retention by concrete curing materials	C156

6.6.2.2 Hardened Concrete Tests. Hardened concrete tests are summarized in Table 12.

Table 12
Hardened Concrete Tests

TEST DESCRIPTION	ASTM DESIGNATION
Air-void content	C457
Capping cylinders	C617
Compressive strength	C39
Accelerated curing and testing of concrete	C684
Splitting tensile strength	C496
Flexural strength	C78
Modulus of elasticity and Poisson's ratio	C469
Creep of concrete in compression	C512
Specific gravity, absorption and voids	C642
Cement content of hardened concrete	C85
Resistance to rapid freezing and thawing	C666
Abrasion resistance to sandblasting	C418
Abrasion resistance of horizontal surfaces	C779
Scaling resistance of concrete exposed to deicing salts	C672
Examining and sampling of concrete in constructions	C823

6.7 Mix Design Process.

6.7.1 Mix Design Procedures. Design concrete in accordance with ACI 211.1, Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete, except as modified by guide specification NFGS-02520. All concrete shall be proportioned by weighing, except when equipment failures or other unusual circumstances necessitate the temporary use of volumetric proportioning. The minimum flexural strength at 28 days shall be 650 pounds per square inch (4.5 MPa).

6.7.2 Design of Lean Concrete Mixtures. Lean concrete mixtures described in Section 5 can be used as a base material. Mix procedures for normal weight concrete given in ACI 211.1 and NFGS-02520 can also be used for the design of lean concrete mixtures. Exceptions to the procedure include the use of a single aggregate instead of a combination of coarse and fine aggregates and a lower cement content than that used for normal weight concrete. The primary considerations in designing a lean concrete mixture are workability, adequate strength and durability. The mixture must be workable and easy to place and consolidate using vibration and standard concrete paving equipment. The mixture must also be cohesive enough to be placed with a slipform paver. Adequate strength and durability are required to withstand traffic loadings and environmental conditions.

Conduct laboratory testing of trial mixes to determine the cement content required to produce the desired workability and strength properties. Typical cement factors range from 200 to 350 pounds per cubic yard (119 to 208 kg per cubic meter) for lean concrete bases. Flexural strengths at 28 days should be between 300 and 400 pounds per square inch (2.1 and 2.8 MPa).

Air entrainment shall be used to improve freeze-thaw resistance and workability. Typical air contents for air-entrained lean concrete mixtures are between 3 and 9 percent. Maintain slumps in the 1- to 3-inch (25 to 76 mm) range when placing lean concrete mixtures. Specifications given in NFGS-02520 for concrete are also applicable for the mixing, transporting, and placement of lean concrete mixtures.

6.8 Hot Weather Considerations.

The following sections provide general information on hot weather considerations.

6.8.1 Effects of High Temperatures. Hot weather is a combination of high air temperature, low relative humidity, and wind velocity which may impair the quality of fresh or hardened concrete. Hot weather can seriously affect the characteristics of plastic concrete. These include:

a) increased water demand,
b) earlier and more rapid slump loss,
c) increased rate of setting,
e) increased chance of plastic cracking,
e) problems in controlling entrained air content.

6.8.5 Curing. The goal of curing is to maintain sufficient water in the concrete to continue hydration and maximize strength gain. To obtain proper curing during hot weather it is important to reduce moisture loss and prevent plastic cracking. Strength gain will stop when sufficient moisture for curing is no longer available. Continuous moist curing is preferred. If moist curing cannot be accomplished, or if it cannot be continued beyond 24 hours, protect the concrete surface for the remainder of the curing period with plastic sheets, membranes, etc. See NFGS-02520 for specifications on curing materials.

6.9 Cold Weather Considerations. The following sections provide general information on cold weather considerations. See ACI 306, Cold Weather Concreting, for additional information on cold weather concreting practices.

6.9.1 Effects of Cold Weather. Reduced temperatures slow down the rate of hydration and the associated gain in strength. To reduce the chance of damaging concrete by early freezing, use air-entrained concrete in cold weather construction. Air-entrained concrete protects the concrete by providing a reservoir for water that is forced from the cement paste during freezing. High early strength gain is desirable in cold weather operations to reduce the length of time that temporary protection is required. To facilitate early strength gain, use a high-early-strength cement (Type III), an insulation layer, or an accelerating admixture.

6.9.2 Heating of Concrete Materials. It is desirable to prepare fresh concrete to a temperature in the range of 50 deg.F (10 deg. C) to 90 deg. F (32 deg.C) for cold weather construction. Methods to maintain concrete temperatures in cold weather include heating aggregates and water before mixing. Aggregates can be heated by circulating steampipes through stockpiles and by covering stockpiles to retain heat. Use Equation (1) to calculate the approximate concrete temperature.

Do not place concrete when the air temperature in the shade falls below 40 deg. F (4 deg. C), or when the concrete, without special protection, is likely to be exposed to freezing temperatures prior to the completion of the designated curing period. See NFGS-02520 for specifications on heating mixing water and aggregates.

6.9.3 Transporting, Placing, and Finishing. During cold weather, place concrete before the mix temperature drops below a temperature range of 60 deg. F (16 deg. C) to 70 deg. F (21 deg. C). See NFGS-02520 for specifications on placing concrete in cold weather.

6.9.4 Curing. Concrete must be protected from rapid temperature drop when curing. Effective means of insulation include straw or hay and commercial insulation blankets. See NFGS-02520 for specifications on curing materials.

The fatigue damage can be accumulated over any number of stress levels (or different aircraft loadings) as indicated by the summation sign. When the fatigue damage proportion reaches 1.0, or 100 percent, a substantial number of slabs will have cracked.

8.2 Thickness Design Inputs. Five key design inputs are needed to determine the required slab thickness.

8.2.1 Design Concrete Flexural Strength. The 28-day third-point loading flexural strength is used for pavement design. The design flexural strength should be as high as practicable and economical but not less than 650 pounds per square inch (4 482 000 Pa). The actual mean flexural strength in the field will be greater than the design flexural strength. The concrete design shall meet the flexural strength requirements in NFGS-02520.

8.2.2 k Value at Top of Base. The determination of the k value on the subgrade and at the top of the base layers is described in Section 5. The value used for design is that obtained at the top of the base. The combined base and subgrade should have a minimum design k value of 200 pounds per cubic inch (5 536 000 kg per cubic meter) to prevent excessive permanent deformation of the subgrade due to slab corner deflections. A base course of sufficient thickness and quality should be used to achieve this modulus. However, in no case should design be based on a k value greater than 500 pounds per cubic inch (13 840 000 kg per cubic meter).

A stabilized base or lean concrete base may be used as a substitute for a granular base course on a 1:1.5 thickness replacement ratio. However, the k value used for design remains the same as that determined at the top of the granular base. The design k value is not increased due to the use of a stabilized or lean concrete base. An unbonded stabilized or lean concrete base does not increase the effective k value greatly due to slippage between the slab and base.

8.2.3 Type and Design Gear Load of Aircraft Using Facility. Pavement thickness design can be determined for a single design aircraft or for a mix of aircraft traffic. Determine the design gear load for a given aircraft by first selecting the design gross aircraft weight. This is normally the maximum gross aircraft weight at departure. Then estimate the design gear load by assuming that 95 percent of the gross weight is carried by the main gears. Design values are given in Tables 13 and 14 in Section 7.

8.2.4 Number of Aircraft Passes. Forecast the total number of passes (not coverages) of each aircraft that is expected to use the pavement feature over its design life. Some general recommendations are provided in Section 7. The "number of passes" is normally the number of departures at the maximum gross aircraft weight. The exception to this is in touchdown areas on runways where the impact due to aircraft performing touch-and-go operations will cause pavement damage. On pavements that are to be used for touch-and-go operations, add the expected number of touch-and-go operations over the design life to the number of departures to arrive at the design traffic passes. Minimum pass levels for design are given in Table 16.

8.2.5 Primary or Secondary Traffic Area. Guidance on determining if the pavement feature is a primary or secondary traffic area is given in Section 7.

8.3 Thickness Design Procedure for a Single Design Aircraft. Use Figures 4 through 8 to determine the concrete slab thickness for single design aircraft. These charts were developed using stresses that were computed using the computer program PCAPAVE developed by the PCA for the interior stress condition.

The design chart for aircraft with single wheel gear is shown in Figure 4 and is used by entering the design flexural strength and the tire load and projecting as shown by the dashed example lines until the required slab thickness is obtained. The design charts shown in Figures 5 through 8 are also entered with the design concrete flexural strength and projecting as shown by the dashed example lines until the required slab thickness is obtained.

The calculated slab thickness is rounded to a whole inch to obtain the design thickness. If the computed thickness is less than or equal to 0.25 inches (6 mm) greater than a whole inch, the thickness is rounded downward (e.g., 10.15 inches (256 mm) is rounded to 10 inches (254 mm)). If the thickness is more than 0.25 inches (6 mm) greater than a whole inch, the thickness is rounded upward (e.g. 10.30 inches is rounded to 11 inches (279 mm)).

This procedure will provide the required slab thickness for a specified type of aircraft when the flexural strength, k value, gear load, tire pressure for single wheel gear aircraft, number of passes and type of design traffic area are specified. The example presented in Table 18 illustrates how to determine the required slab thickness using the design chart for a single design aircraft.

If design charts are needed for any aircraft not included in Figures 4 through 8, use the PCA computer program PCAPAVE through the NAVFAC Engineering and Design Division Time Sharing Computer Library along with the fatigue life data given in Table 17 to develop a required pavement design.

Table 18
Design Example for a Single Design Aircraft

Aircraft - C-141
Design gear load - 155,000 pounds (70 300 kg)
Design flexural strength - 650 pounds per square inch (4.5 MPa)
Effective k value at top of base course - 200 pounds per cubic inch (5 536 000 kg/m^3)
Total departures over 20-year design life - 25,000
Traffic area - primary taxiway (channelized traffic)
Required slab thickness - 13.4 inches (340 mm) (from Figure 7)
This thickness should be rounded up to 14.0 inches (356 mm)

Section 9: JOINT DESIGN FOR NON-REINFORCED CONCRETE PAVEMENT

9.1 Introduction. The following sections describe the joint designs to be used for non-reinforced concrete pavements. See NFGS-02522, Joints, Reinforcement and Mooring Eyes in Concrete Pavements, for guide specifications on joints and reinforcement.

9.2 Basis for Design. Use joints to:

a) limit curling and warping stresses in the pavement which are due to temperature and moisture gradients through the slab,
b) prevent and control cracking due to volume changes in the concrete,
c) prevent damage to immovable structures,
d) facilitate construction.

9.3 Types of Joints and Uses.

9.3.1 Expansion Joints. Expansion joints allow for the expansion of the pavement and the reduction of high compressive stresses at critical locations in the concrete pavement in hot weather. Expansion joints are placed the full depth of the slab. Use expansion joints at all intersections of pavements with fixed structures, at nonperpendicular pavement intersections and between existing and new concrete pavements when the joints in the adjacent slabs are not aligned. Expansion joints are not otherwise required within the non-reinforced concrete pavement. See Figure 14 for expansion joint details.

9.3.2 Contraction (Weakened Plane) Joints. Use contraction joints to: (1) control cracking in the pavement due to volume changes resulting from a temperature decrease or a moisture decrease, and (2) limit curling and warping stresses from temperature and moisture gradients in the pavement. Form contraction joints in concrete by partial depth sawing or by installing sawable inserts. The saw cut joint or formed groove provides a weakened plane which will crack through the full slab depth during shrinkage and contraction of the concrete as it cures. Contraction joints are required in the transverse direction and also in the longitudinal direction depending upon slab thickness and spacing of the construction joints. See Figure 15 for contraction joint details.

9.3.3 Construction Joints. Construction joints are used between paving lanes or when abutting slabs are placed at different times. Longitudinal and transverse construction joints may be required. Transverse construction joints will be required when it is necessary to stop concrete placement for a length of time sufficient to allow the concrete to begin to set. Longitudinal construction joints are generally spaced 20 to 50 feet (6 to 15 m) apart depending on the construction equipment.

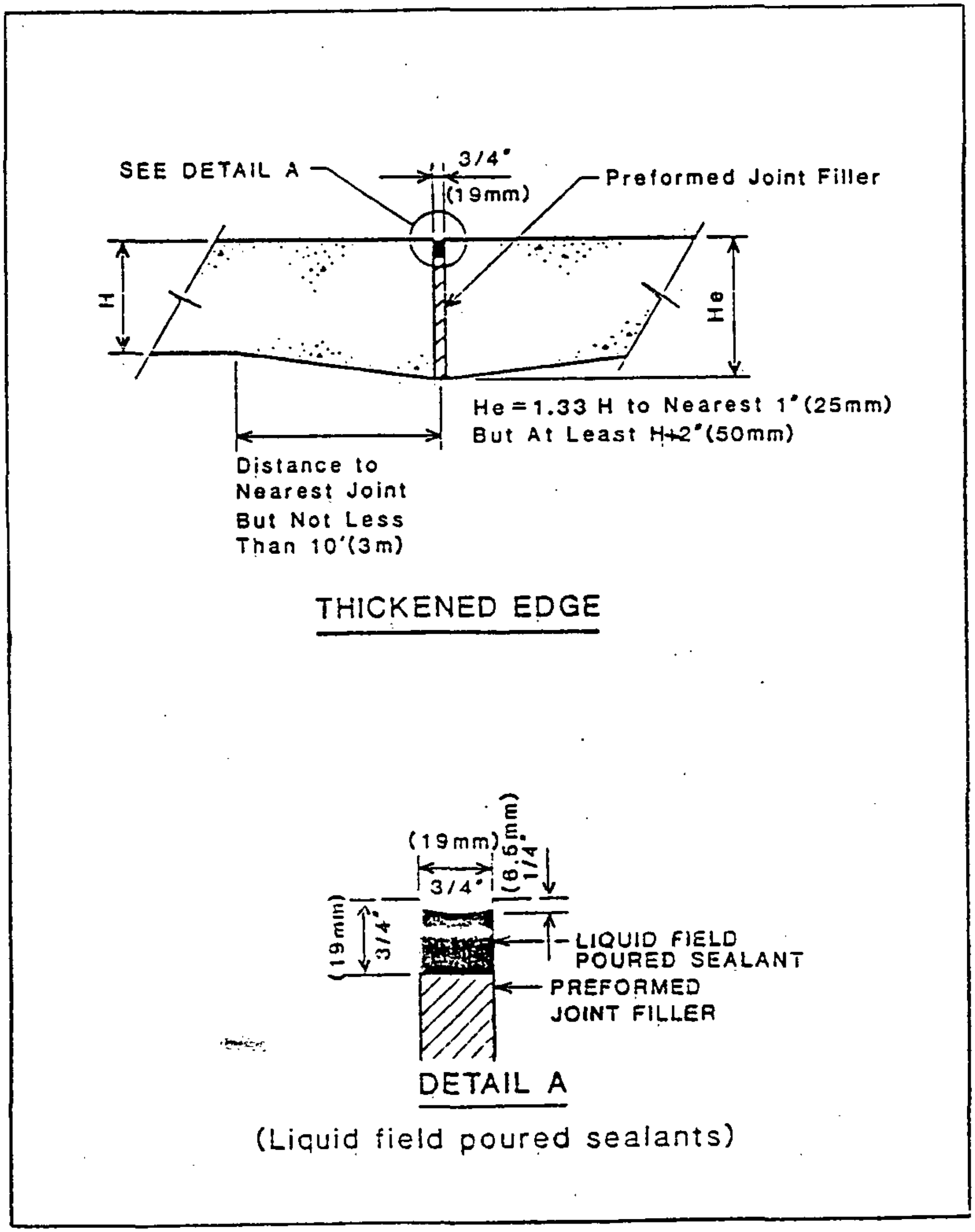

Figure 14
Expansion Joint Details

9.5.2 Dowel Bars. Dowel bars are used to provide load transfer and prevent excessive vertical displacements of adjacent slabs. Dowels are to be avoided for new construction because of a history of quality control problems (such as poor alignment). There are some situations where the use of dowels is appropriate, such as for creating load transfer where tying in to existing pavements.

9.5.3 Keyways. Keyways may be used to provide load transfer along longitudinal construction joints. Do not use keyways in pavements less than 9 inches (229 mm) thick because the small keyways have limited strength and may crack under heavy aircraft loadings. Small keyways may also be difficult to construct because coarse aggregate may not be able to enter the key and segregation of the material may occur. Keyways may be used for pavements 9 inches (229 mm) thick and greater. However, keyway failures have caused serious problems.

9.5.4 Stabilized Base. A stabilized base can be used to improve load transfer effectiveness by reducing joint deflections through increased support across a joint. Use a stabilized base for all pavements less than 9 inches (229 mm) thick to provide improved load transfer and lower deflections and stresses. A stabilized base may also be used for pavements greater than 9 inches (229 mm) thick to provide additional load transfer. Where thickened edge joints are used, the stabilized base is not required.

9.6 Joint Sealants. Joint sealants are used to provide a seal to reduce infiltration of water and incompressibles. An effective joint seal will help retard and reduce distress related to free water and incompressibles, such as pumping, spalling, faulting, and corrosion of mechanical load transfer devices. See NFGS-02522 for specifications on joint sealing compounds.

Several pavement areas require fuel-resistant or blast-resistant joint sealants. Use jet fuel resistant sealants for all aprons. Use blast resistant sealants for the first 1000 feet (305 m) of runways and exits at runway ends. Use sealing compounds meeting Federal Specification SS-S-1401C, Sealant, Joint, Non-Jet-Fuel-Resistant, Hot-Applied, For Portland Cement and Asphalt Pavements, for taxiways and runway interiors. Specific pavement areas are detailed in NFGS-02522.

9.6.1 Types of Sealant Materials. The three major types of sealant materials are (1) field poured, hot-applied; (2) field poured, cold applied; and (3) preformed compression seals. These materials may be jet fuel resistant (tar based) or non-jet fuel resistant (typically asphalt based).

9.6.1.1 Field Poured, Hot Applied. This group of sealants includes rubberized asphalt sealant and rubberized tar sealant. Rubberized asphalt joint sealants must meet Federal Specification SS-S-1401C and may be used in the areas designated in NFGS-02522. Rubberized tar sealants must meet Federal Specification SS-S-1614A, Sealant, Joint, Jet-Fuel-Resistant, Hot-Applied, For Portland Cement and Tar Concrete Pavements.

9.6.1.2 Field Poured, Cold Applied. These are two-component, polymer-type, cold-applied heat and jet fuel-resistant joint sealants. These sealants must meet Federal Specification SS-S-200E, Sealant, Joint, Two-Component, Jet-Blast-Resistant, Cold-Applied, For Portland Cement Concrete Pavements.

9.6.1.3 Preformed Compression Seals. The most common type of preformed compression seal is the neoprene compression seal. Neoprene compression seals must satisfy ASTM D2628. Preformed compression seals may be used in the areas designated in NFGS-02522. Preformed compression seals are designed to be in compression for their entire life. There is no bond between the compression seal and the sidewalls of the joint to sustain tension.

9.6.2 Joint Reservoir Design. The joint reservoir must be properly designed so that the joint sealant can withstand compressive and tensile strains.

9.6.2.1 Field Poured Sealants. The shape factor, which is defined as the ratio of the depth of the sealant to the width of the joint, should be between 1.0 and 1.5. Dimensions of the joint sealant and reservoir are shown in Figure 15. A backer rod or bond breaking tape must be used to help obtain a proper shape factor and to prevent the joint sealant from bonding to the bottom of the joint reservoir. Most field-poured liquid joint sealants can withstand strains of approximately 25 percent of their original width. Joint reservoir and sealant dimensions shown in Figure 15 are based on a slab size of 12.5 feet (3.8 m) by 15.0 feet (4.6 m).

9.6.2.2 Preformed Compression Seals. The reservoir width for preformed compression seals must be designed to keep the sealant in compression at all times. The depth of the reservoir must exceed the depth of the seal, but is not related directly to the width of the joint. The width of the compression seal should be approximately twice the width of the joint. The limits on the compression seal are normally 20 percent minimum and 50 percent maximum compression strain of the original sealant width. For example, the working range of a 1-inch (25 mm) wide neoprene compression seal is from 0.5 to 0.8 inches (12 to 20 mm).

If the seal is subjected to compression greater than the 50 percent level for extended periods of time, the seal may take a compression set, and the webs may bond to each other. If this happens, the seal will not open as the joint opens, and the seal will no longer be effective.

The joint dimensions for the standard size slab are shown in Figure 15. Design sealant dimensions based on the actual joint spacing. Choose preformed neoprene compression seal dimensions so that the working range of the joint is within the working range of the sealant.

9.7 Tiedown Mooring Eyes. Tiedown mooring eyes are required in the center of each slab over the entire aircraft parking apron except on peripheral taxi lanes. See Figure 18 for mooring eye details.

Section 11: STRENGTHENING OF RIGID PAVEMENTS

11.1 General. Strengthening of a pavement increases its load-bearing capacity, and is accomplished with an overlay of either asphalt concrete or Portland cement concrete. Airfield pavements require strengthening to support heavier and/or more numerous aircraft loadings than those for which they were originally designed. Strengthening is also performed on pavements which have been damaged by a combination of traffic loadings and climatic influences to the extent that they can no longer support the traffic expected to use them. This section provides guidance and criteria for the design of flexible and rigid overlays of rigid airfield pavements to increase structural capacity. Functional overlays, those which improve operational condition of the airfield pavement but do not significantly increase its structural capacity, are also discussed.

11.2 Rehabilitation Alternative Selection. A structural improvement such as an overlay should be performed only after it has been determined that the pavement possesses a structural deficiency for which an overlay is an appropriate and cost-effective method of rehabilitation. The major steps in the overall pavement rehabilitation alternative selection process are:

a) Office data collection: for pavement age, design, materials and soils properties at the time of construction, traffic data, climate records, and maintenance history.

b) Field and lab data collection and testing: for present structural and functional condition, surface and subsurface drainage conditions, vertical grade, and in situ materials and soils properties.

c) Definition of the pavement deterioration problem: in terms of the types of deterioration present and the mechanisms responsible for the deterioration.

d) Development of feasible rehabilitation alternatives: which both repair the existing distress and prevent its recurrence, as much as possible, by effectively addressing the causes of the deterioration.

e) Selection of the preferred alternative: which is the most appropriate and most cost-effective method of rehabilitation for the pavement, given existing constraints (e.g., funding, allowable closure time).

11.2.1 Pavement Evaluation Procedure. A comprehensive evaluation of the present condition of the pavement must be performed to determine an accurate and complete definition of the pavement deterioration problem.

11.2.1.1 Condition Survey. Use the Pavement Condition Index (PCI) rating procedure for performing the visual distress survey and condition rating of Navy and Marine Corps airfield pavements. The PCI procedure has been adopted by the U. S. Navy for airfield pavement condition rating.

a) Description of PCI. The PCI is a number from 0 to 100 which reflects the structural and functional condition of a pavement as it would be rated subjectively by a panel of experienced airfield pavement engineers. Its scale and associated condition ratings are shown in Figure 19. The PCI is calculated from data collected during a visual distress survey in which pavement distresses are quantified by type, amount, and severity. The mean PCI can be computed for any individual pavement. Procedures have been developed for performing the distress survey either by sampling a portion of the pavement surface or by inspecting the entire pavement area.

b) Overall Condition. The mean PCI of a pavement section describes its overall condition (e.g., fair, good, etc.) and thus is an indicator of the level of repair or rehabilitation work needed. The following are general guidelines for the level of repair or rehabilitation work that may be expected to be most cost-effective for a pavement with a given PCI:

Current PCI	Most Cost-Effective Rehabilitation (Within Next Two Years)
100 to 70	Preventive maintenance and restoration, including joint/crack sealing, undersealing (filling voids), slab replacement, full-depth repair, partial-depth spall repair.
69 to 40	Most cost-effective rehabilitation may range from preventive maintenance to major rehabilitation. Decision requires an engineering and a life-cycle cost analysis.
39 to 25	Major rehabilitation including overlays with or without keel replacement.
24 to 0	Major reconstruction (overlay is possible with extensive pre-overlay repair).

Note: There are exceptions to these guidelines, such as when a change in mission aircraft necessitates an increase in structural capacity regardless of the PCI value.

If an unbonded concrete overlay is being designed, the flexural stress does not need to be adjusted for the load transfer efficiency of the existing slabs unless the joints in the overlay will be matched with the joints in the existing slab. For mismatched joints, the load transfer adjustment factor is one (1.0) and the critical stress is recorded in Column 8 as the same value as the interior stress from Column 6.

11.7.1.3 Fatigue Life Consumption. The fatigue life consumption is determined as described in Section 8 of this manual. The fatigue life consumed must be less than 1.0 (or 100 percent). The process of selecting a slab thickness, determining the flexural stress, and calculating the fatigue life consumption is repeated until the slab thickness which corresponds to an acceptable value for Miner's damage (less than 1.00 or 100 percent) is determined.

11.7.1.4 Determination of C_r. The condition of the existing concrete slab is taken into consideration during the design of the concrete overlay. The existing concrete pavement may have suffered deterioration from climate and material durability as well as fatigue damage from traffic loadings throughout the life of the pavement. To account for these factors, a condition factor, C_r, is used to evaluate the structural condition of the existing concrete slab. The condition factor, C_r, is determined using the results of the PCI survey. When determining C_r the only distresses considered are those associated with structural loading. These include:

a) longitudinal, transverse, and diagonal cracks of medium to high severity,
b) corner breaks of any severity,
c) all large patches of load-associated failures,
d) pumping,
e) settlement or faulting of any severity,
f) shattered slabs of any severity,
g)certain types of joint spalls believed to be load-associated (e.g., keyway failures).

The PCI_{STR} (PCI based on structural distress only) is then calculated using only these structural distresses.

The value of C_r ranges from 0.35 to 1.0. A C_r of 0.35 corresponds to a condition where approximately 60 percent of the concrete slabs are shattered with a severity level of medium, or 50 percent of the slabs have high severity longitudinal, transverse, or diagonal cracks. A C_r of 1.0 corresponds to a condition where the pavement is in very good condition with little or no structural cracking.

A correlation between PCI computed from structural distress only (PCI_{STR}) and C_r is shown in Figure 25.

11.7.1.5 Determination of Overlay Thickness (T_o). The required thickness of the concrete overlay (T_o) can be computed using Equation (5) and the appropriate exponent for the type of overlay. An exponent of 1.0 should be used for bonded concrete overlays, 1.4 for partially bonded concrete overlays, and 2.0 for unbonded overlays. The calculated thickness (T_o) is rounded to the nearest whole inch to obtain the design concrete overlay thickness. If the computed overlay thickness is less than or equal to 0.25 inches (6 mm) greater than a whole inch, the thickness is rounded downward (e.g., 10.15 inches is rounded to 10 inches). If the thickness is more than 0.25 inches (6 mm) greater than a whole inch, the thickness is rounded upward (e.g., 10.30 inches is rounded to 11 inches).

11.7.2 Asphalt Concrete Overlay Thickness Design. The required asphalt concrete overlay thickness is obtained using Equation (6). The procedure described in sub-paragraph 7.a. above should first be used to determine the critical flexural stress and the required single slab thickness (T_n) for a new bonded concrete overlay design. Then this thickness is adjusted as described below to obtain the required thickness of asphalt overlay (T_o).

11.7.2.1 Determination of C_b. The structural condition of the existing concrete slab is taken into consideration by use of a condition factor, Cb. This factor ranges from 0.50 to 1.0. A C_b value of 1.0 indicates that the existing slabs are in excellent condition and contain none or only nominal cracking. A C_b value of 0.50 indicates that the existing slabs contain multiple cracking.

A relationship between PCI_{STR} and C_b is shown in Figure 26. PCI_{STR} is computed from the existing structural distress in exactly the same manner as described for the rigid pavement condition factor.

11.7.2.2 Determination of F. The required single slab thickness for a new design (T_n) is modified by a factor "F" which controls the amount of cracking which is allowed to occur in the base concrete slab. The "F" factor essentially reduces the required single slab thickness which will increase the critical stress. This reduction in thickness is allowed because an asphalt overlay is allowed to crack and deflect more than a rigid pavement. More cracking is allowed in an asphalt overlay because the asphalt overlay can conform to greater deflections than a rigid pavement without excessive spalling.

The "F" factor decreases as the effective k value on top of the base course increases. Thus, as the effective k value increases, the thickness of the asphalt concrete overlay decreases. This reduction in thickness is allowed because the increased slab support provides a stable foundation that reduces movements and deflections when the existing concrete slab begins to crack. Determine the appropriate "F" factor from Figure 27.

11.9.2 Asphalt Concrete Overlay Design Example. An asphalt concrete overlay is to be designed for the same runway and traffic conditions presented in paragraph a. above. Table 23 summarizes the calculations to determine the required single slab thickness for the primary traffic area. As summarized in the previous example, a single slab thickness of 13.5 inches (345 mm) is needed for the primary traffic areas. The "F" factor is determined to be 0.84 from figure 27 for an effective k value of 250 pci.

The condition factor C_b is determined to be 0.90 for a PCI_{STR} of 70 from Figure 26.

The required asphalt concrete overlay thickness for channelized (primary) traffic areas is calculated as follows:

$$T_o = 2.5\ (FT_n - C_bT_e)$$

$$T_o = 2.5\ (0.84*13.5 - 0.90*9.0)$$

T_o = 8.1 inches (rounded to 8.0 inches (203 mm)

The required asphalt concrete overlay thickness for unchannelized (secondary) traffic areas is calculated as above and is determined to be 8.1 inches (229 mm). This is rounded to 8.0 inches (203 mm).

SECTION 12: AIRCRAFT AND ENGINE RESTRAINT SYSTEM PROOF LOAD TESTING REQUIREMENTS

12.1 Purpose. This inspection and test is to determine that all component parts of land-based aircraft and engine restraint systems are in a safe and properly maintained operating condition.

12.2 Inspection and Test Frequency. The system consists of a restraint fitting (Type XIII, XI-A, etc.) fastened to a steel embedment which is anchored in the concrete foundation. Load test the system at the following intervals:

a) after initial installation. Load test new installations prior to operational use;
b) annually;
c) after repair and/or replacement of any of the components (i.e., restraint fittings, bolts, etc.);
d) due to visual signs of deterioration.

After completion of a proof test, NAVAIR requires a non-destructive inspection (NDI) of all restraint fittings and any operational hardware used during the test. When a restraint fitting is removed for NDI after a proof test and then re-installed, it is not necessary to re-test the fitting at that time. Refer to NAVAIR Technical Manual 17-1-537. Aircraft Securing and Handling Procedures with Organizational, Intermediate, and Depot Maintenance For Aircraft Restraining Devices and Related Components for information on NDI requirements.

12.3 Description of System. The proof load system (Figure 28) generally consists of: a hydraulic system for applying the load, which includes a hand pump and a single acting pull cylinder; special structural fixtures which interface with the proof load test fitting; and a chain assembly, turnbuckle and hardware to interface with the restraint fitting to be tested.

12.4 Proof Loads

a) Insure the proof load tests for Jet Engine Test Cells and Unabated Power Check Facility restraint systems are in accordance with Table 25. Use test procedures which gradually increase the load in increments as follows:

(1) Single XI-A ("+" configuration) - Proof-test load = 30,000 lbs.

(a) 10,000 lbs - hold for one minute
(b) 20,000 lbs - hold for one minute
(c) 25,000 lbs - hold for one minute
(d) 30,000 lbs - hold for ten minutes

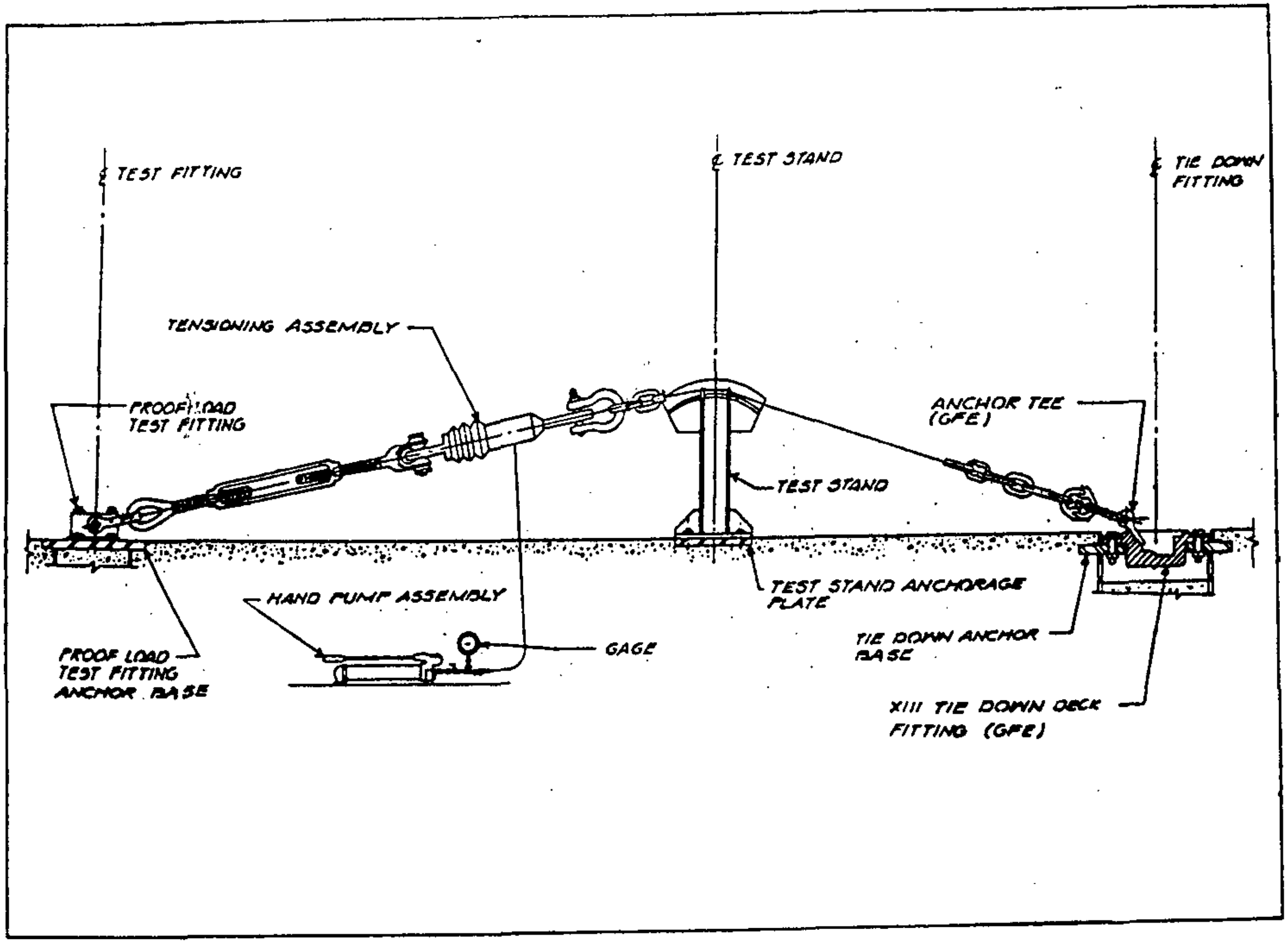

Figure 28
Proof Load Test System

(2) Single XI-A ("X" configuration) and Single F-2 - Proof-test load - 45,000 lbs.

(a) 10,000 lbs. - hold for one minute
(b) 30,000 lbs. - hold for one minute
(c) 40,000 lbs. - hold for one minute
(d) 45,000 lbs. - hold for ten minutes

(3) Dual XI-A ("X" or "+" configuration), Dual F2 and Dual XIII - Proof-test load - 60,000 lbs.

(a) 10,000 lbs. - hold for one minute
(b) 30,000 lbs. - hold for one minute
(c) 40,000 lbs. - hold for one minute
(d) 50,000 lbs. - hold for one minute
(e) 60,000 lbs. - hold for ten minutes

(4) Single XIII and Single F1-Proof-test load - 90,000 lbs.

(a) 20,000 lbs. - hold for one minute
(b) 45,000 lbs. - hold for one minute
(c) 70,000 lbs. - hold for one minute
(d) 90,000 lbs. - hold for ten minutes

b) Insure the proof test load for Aircraft Acoustical Enclosure (Hush House) restraint systems is in accordance with Table 26. Use test procedures which gradually increase the load in increments as follows:

(1) Single XIII, Single F1 and Hybrid - Proof-test load - 60,000 lbs.

(a) 10,000 lbs. - hold for one minute
(b) 30,000 lbs. - hold for one minute
(c) 40,000 lbs. - hold for one minute
(d) 50,000 lbs. - hold for one minute
(e) 60,000 lbs. - hold for ten minutes

12.5 Inspection and Test Procedures. Induce tension in the chain assembly by slowly pumping the hydraulic hand pump until the assembly is taunt. When the load is applied, keep personnel clear of the immediate test area.

Increase the load gradually in increments. After each load increment and when the final proof load is reached, the hydraulic safety valve should be closed and held as indicated above. The restraint system should be remotely observed during the loading phases. If any indication of cracking or deformation is apparent, the load should be gradually released and the test terminated.

After the loading phase is completed and the load released, the restraint system components (fittings, bolts, anchor base and concrete in the immediate area) should be inspected for signs of distress. Fittings and anchors showing such distress are unacceptable and shall be replaced.

12.6 Scheduling and Performance Responsibility. Scheduling of inspections and tests is the responsibility of the Commanding Officer of a Public Works Center or the activity Public Works Officer. Inspection and tests shall be made by qualified personnel on the cognizant activity's rolls, except:

a) Where inspection responsibility has been assigned to the Commanding Officer of a Public Works Center.

b) Where Commanding Officers of major or lead activities are responsible for performing the maintenance of Public Works and Public Utilities at adjacent activities.

c) Where it may be impracticable to employ qualified personnel for such inspections and tests because of the limited workload. In such situations, request assistance in obtaining inspection and test services from the appropriate Engineering Field Division Commander/Commanding Officer. The Engineering Field Division Commander/Commanding Officer shall arrange for the performance of these inspection and test services by an activity, having qualified personnel, located near the requesting activity, or by contract.

d) The static pull test indicated in paragraph 12.5 should be performed only under the guidance and supervision of an engineer.

e) Perform Structural Control Inspections by qualified activity personnel, or by contract.

12.7 Inspection and Test Results

12.7.1 Inspections and Tests by Activity. Promptly initiate necessary action to correct deficiencies found during inspections and tests.

12.7.2 Inspections and Tests by Engineering Field Division. This covers inspections and tests performed by EFD personnel or by contract.

a) Staff furnish the activity a list of deficiencies found and required corrective action.

b) Contract furnish the activity a copy of the contractor's deficiencies report.

12.7.3 Action on Deficiencies. If deficiencies found by inspections and tests are such that the Aircraft Power Check Facility is considered unsafe, the Facility should be secured until such time as the deficiencies are corrected.

12.8 Form. The Facility Condition Report (as detailed in MO-322, Inspection of Shore Facilities, Vol. I) may be used for recording deficiencies. It should be dated and signed by cognizant personnel responsible for conduct of the inspection and test. A copy of the Inspector's Report should be forwarded to the activity's Aircraft Intermediate Maintenance Department.

Table 25
Jet Engine Test Cells and Unabated Power Check Facility
Restraint System Proof Lands/Pull Angles

Deck Fitting(s) Type	Maximum Safe Working Load Pounds	In-Service Usage	Proof Test-Load Pounds	Proof-Test-Angle Degrees	NOTE
Single XI-A ("+") Configuration	15,000	In-Airframe Run-up-and/ or Out of Airframe Engine Run-up	30,000	15	(1)
Single XI-A ("X") Configuration	22,500	In-Airframe Run-up and/ or-Out of Airframe Engine Run-up	45,000	15	(2)
Dual XI-A ("X" or "+") Configuration	30,000	Out-of-Airframe Engine Run-up	60,000	30	(3)
Single F2	22,500	In-Airframe Run-up and/ or-Out of Airframe Engine Run-up	45,000	15	(2)
Dual F2	30,000	Out-of-Airframe Engine Run-up	60,000	30	(3)
Single XIII	45,000	In-Airframe Run-up and/ or-Out of Airframe Engine Run-up	90,000	15	(4)

Table 25 (Cont.)
Jet Engine Test Cells and Unabated Power Check Facility
Restraint System Proof Lands/Pull Angles

Deck Fitting(s) Type	Maximum Safe Working Load Pounds	In-Service Usage	Proof Test-Load Pounds	Proof-Test-Angle Degrees	NOTE
Dual XIII	30,000	Out-of-Airframe Engine Run-up	60,000	30	(3)
Single F1	45,000	In-Airframe Run-up	90,000	15	(4)

NOTES:

(1) A 30,000 pount proof-load-test of a single, "+" configured Type XI-A fitting certifies a site for a maximum safe working load of 15,000 pounds.

Simultaneous, dual-engine, maximum testing (military and/or afterburner) is prohibited for all aircraft. All aircraft and engine testing exceeding 15,000 pounds of thrust is also prohibited.

(2) A 45,000 proof-load-test of a single fitting certifies a site to test all aircraft (except F-14A+/D) and all engines (except F110-GE-400). This does not include simultaneous, dual engine afterburner testing of F-4, F-14A and F/A-18 aircraft.

(3) An 60,000 pound proof-load-test on a dual set of fittings certifies a site to test all engines.

(4) A 90,000 pound proof-load-test on a single fitting certifies a site to test all aircraft (excluding simultaneous, dual-engine afterburner testing of F-14A+/D aircraft) and all engines.

Table 26
Aircraft Acoustical Enclosure Restraint System
Proof Loads/Pull Angles

Deck Fitting(s) Type	Maximum Safe Working Load Pounds	In-Service Usage	Proof Test-Load Pounds	Proof-Test-Angle Degrees	NOTE
Single XIII	30,000	In-Airframe Run-up	60,000	15	(1)
Single F1	30,000	In-Airframe Run-up	60,000	15	(1)
Hybrid	30,000	In-Airframe Run-up	60,000	15	(1) & (2)

NOTES:

(1) Simultaneous, Dual-engine, afterburner testing is prohibited for all aircraft.

(2) Applies to non-standard fittings at NAS Patuxent River, MD. and NAS Jacksonville, FL.

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NOTE: THE FOLLOWING REFERENCED DOCUMENTS FORM A PART OF THIS HANDBOOK TO THE EXTENT SPECIFIED HEREIN. USERS OF THIS HANDBOOK SHOULD REFER TO THE LATEST REVISIONS OF CITED DOCUMENTS UNLESS OTHERWISE DIRECTED.

FEDERAL SPECIFICATIONS, MILITARY HANDBOOKS, AND NAVFAC GUIDE SPECIFICATIONS: Available from the Defense Printing Service, Standardization Document Order Desk, Building 4D, 700 Robbins Avenue, Philadelphia, PA 19111-5094

FEDERAL SPECIFICATIONS

SS-S-200E	Sealant, Joint, Two-Component, Jet-Blast Resistant, Cold-Applied, for Portland Cement Concrete Pavement
SS-S-1401C	Sealant, Joint, Non-Jet-Fuel-Resistant, Hot-Applied, for Portland Cement and Asphalt Pavement
SS-S-1614A	Sealant, Joint, Jet-Fuel-Resistant Hot-Applied for Portland Cement and Tar Concrete Pavement

MILITARY HANDBOOKS

MIL-HDBK-1001/3	Architectural Acoustics
MIL-HDBK-1021/1	Airfield Geometric Design
MIL-HDBK-1021/2	General Concepts for Pavement Design
MIL-HDBK-1023/1	Airfield Lighting
MIL-HDBK-1028/1	Aircraft Maintenance Facilities

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NFGS-02511	Bituminous Hot Mix Pavement
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C70	Test Method for Surface Moisture in Fine Aggregate
C78	Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
C85	Test Method of Cement Content of Hardened Portland Cement Concrete
C87	Effect of Organic Impurities in Fine Aggregate on Strength of Mortar
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