DIVISION 500 – STRUCTURES

SECTION 501 – TEMPORARY BRIDGE

501.1 – GENERAL

The Contract Administrator inspects the construction, maintenance, and removal of a temporary bridge and helps coordinate the activities of various personnel in providing traffic safety.

Figure 500 – 1: Temporary Highway Bridge Installation

501.3 – CONSTRUCTION OPERATIONS

A. Approval of Plans

The Contractor will provide the Contract Administrator with detailed plans designed and stamped by a licensed professional engineer for the temporary bridge item. The Contract Administrator will forward plans for temporary bridge structures to the Construction Bureau office for approval and distribution.

B. Inspection

The Contract Administrator must inspect the work to ensure that the Contractor builds the bridge in accordance with the submitted plans. Particular attention should be focused upon preparation of the abutment foundations and obtaining a thoroughly compacted foundation. The Contractor is responsible for the maintenance of the temporary bridge throughout its use. Payment of this item should consist of partial estimates for the three stages (construction, maintenance, and removal), rather than 100% upon erection.

C. Opening Bridge to Traffic

Signs and Striping: Signing for detours onto the temporary bridge shall be provided and maintained by the Contractor under Item 619.1 Maintenance of Traffic. Since each temporary bridge location and its approaches are individually designed, the plans and special provisions must be checked for lighting, special barricades, flashing arrows, etc., which might be required in addition to normal signing. If the approaches are to be paved, striping should be completed and existing lines on the approaches removed as necessary to ensure that the traveling public does not get confused.

Notification: The Contract Administrator should notify the NHDOT Public Information officer, the NHDOT Wide Load Permitting office, the Construction office, and local authorities at least one week prior to opening a temporary bridge and again prior to changing traffic to the new alignment. Contact information for NHDOT offices may be found in *Section 900* of this Manual.

SECTION 502 – REMOVAL OF EXISTING BRIDGE STRUCTURES

502.1 – GENERAL

This item consists of the removal and satisfactory recycling or disposal of the existing bridge structure. Particular care should be taken by the Contract Administrator to determine the exact limits of removal as denoted on the plans prior to commencing the operation. Considerable misunderstanding can result when limit lines are not clearly designated on the plans. A common example of this situation is in determining whether a wing wall is considered a portion of the abutment structure or a separate retaining wall structure. Clarification of removal limits at an early date will eliminate considerable confusion.

502.3 – CONSTRUCTION OPERATIONS

The Contractor shall dismantle the existing structure in a manner that will not cause damage to persons or property, nor interfere with the traveling public. A removal plan shall be submitted and accepted before any work may commence.

Prior to removal operations by the Contractor, the Contract Administrator can expedite the removal operation by ensuring that all utilities suspected of being in the vicinity of the structure have been identified, relocated, or terminated. This should be discussed at the pre–construction conference.

Figure 500 – 2: Bridge Demolition and Removal Operations

In the event that the Contractor chooses to use explosives in the removal operation, special consideration must be taken to ensure that there will be no damage to the new work. If the Contract Administrator is in doubt as to the effects of the blasting, the District Construction Engineer should be contacted for a further study of the operation.

For removal operations adjacent to waterways, special attention should be paid to applicable regulations of the NH Division of Water Supply and Pollution Control as detailed in *Section 107 Legal Relations and Responsibility to Public* of the Standard Specifications.

Because older existing structural steel members may be coated with lead paint, the Contractor should ensure that measures are taken to minimize the further contamination to the site. Lead paint that is firmly adhered to the surface of structural members is not considered a hazard. However, operations such as cutting, scaling, or blasting will disturb lead paint coatings, or the paint may be loose or peeling due to exposure, with lead paint chips littering the ground beneath the bridge. The Contractor must follow all regulatory safety procedures when removing structures with regards to lead exposure and Contractor's demolition plan should clearly discuss the proposed methods for handling any waste produced.

SECTION 503 – COFFERDAMS

503.1 – GENERAL

A cofferdam is a form of shoring, generally of a temporary nature, that is constructed for the purpose of keeping water and earth out of a structure excavation. The type of cofferdam usually depends on the water conditions encountered. Small streams can usually be handled with an earth embankment or diversion channel. Deeper rivers will require cribs, sheeting, or caissons to hold back water and earth pressures and provide a safe work area on or below the river bottom. Cofferdams shall be as watertight as necessary to permit de–watering by pumping and shall provide a reasonably dry work area for concrete masonry operations.

Figure 500 – 3: Cofferdam under Construction

A plan must be submitted for documentation prior to the start of cofferdam work. It is the Contract Administrator's duty to ensure conformance with this plan. If changes are made in the cofferdam installation, an as–built set of plans, stamped by a New Hampshire Registered Professional Engineer shall be submitted.

503.3 – CONSTRUCTION OPERATIONS

A. Temporary Diversion Channels

Size and Location: The size of diversionary channels should be sufficient to carry peak seasonal flow. Channels should be located where economically feasible, taking precautions to minimize permanent damage to nearby trees, vegetation, and wildlife.

Water Pollution and Erosion Control: The Contract Administrator and Contractor must become familiar with all instructions concerning water pollution and erosion contained in the contract documents. The Contractor should be made aware of practices to minimize pollution, such as excavating channels in the dry, constructing stone lining or other erosion protective measures, and allowing sufficient time for settlement prior to diverting the stream.

B. Embankment Cofferdams

"Puddle" Cofferdams: For excavations less than 10 ft deep in stable soils, a satisfactory work area often can be provided by using the excavated material in the construction of a dike.

Embankment Cofferdams: For excavations deeper than 10 ft, a more sophisticated construction technique is required. The size of the embankment depends on the distance between depth of excavation and anticipated high water level subject to the angle of repose of the fill material. It is advisable to protect the embankment against current erosion and subsequent water pollution by placing ledge and boulders around the perimeter. Usually, stone slope protection can be placed concurrently with the construction of the earth embankment. Also, at times, soil conditions necessitate the construction of an impermeable core in the embankment.

C. Sheeting Cofferdams

Sheeting Cofferdams: Sheeting is often used in cofferdam construction in deep water and heavy current conditions, for sign structure foundations, and adjacent to existing highway structures and railroad tracks. Construction usually begins by making a frame that, apart from being a primary structural member, acts as a template that holds the sheets to the proper plan dimensions during their driving.

The Contract Administrator should instruct the Contractor to make the initial alignment of the frame as accurate as possible so that any minor movement of the sheets during driving will be of little consequence. The frame must be aligned when it is horizontal and the sheets should be driven plumb for the best results. Should the sheeting travel or lean excessively, it may be necessary to extract the sheets and re–drive or increase the size of the cofferdam.

Care should be taken to drive the sheets well below the bottom elevation of the excavation (toe–in). The Contract Administrator can put a grade on the frame to be used as a reference in determining the tip elevation of the sheets. These elevations should be checked before computation of the tip elevation and they should never be used as a bench mark for future work.

Where the cofferdam is to be the form for a seal pour, additional care should be taken with the alignment of the frame and the driving of the sheets to ensure that the minimum concrete dimensions (inside the projections of channel and Z type sheets) as shown on the plans are maintained. An adequate sump must be provided outside of the forms and inside the cofferdam to allow pumping with minimal seepage through the forms.

Timber Sheet Pile Driving Characteristics: A single or double row of timber sheet piling is sometimes used if only an earth bank is to be supported. Where water tightness is desired, or if earth pressures are large, some form of tongue and groove sheeting is preferable. Triple sheeting withstands driving better than single planks because defects cannot extend through the entire sheeting, and warping is minimized.

Figure 500 – 4: Installing Timber Sheet Pilings

In placing timber sheeting, the tongue should always lead. That is, the groove of the timber being driven should be sliding down over the tongue of the previously driven timber in order to prevent clogging in the groove.

Steel Sheet Pile Driving Characteristics: Interlocking steel sheet piling may be driven by one of two basic methods.

- Method #1 – Driving individual piles: A single pile or pair of piles is driven at one time. The leads should be vertical and stable with the hammer centered over the neutral axis of the pile. Driving piles in pairs generally makes guiding easier. This method is particularly advantageous when a stable and level foundation can be provided for the pile driving equipment. The best practice in driving piling is for the ball end to lead, which prevents soil from becoming trapped in the interlock.

- Method #2 – Continuously driving a preassembled panel of piles: Piling is assembled in wall form first, and driven continuously along the line. It is necessary to be able to set the piling with both axes plumb and hold the hammer rigid. If the Contractor uses swinging leads, the leads and hammer should be held securely in the same vertical plane. Any vibration in the hammer or the pile will result in the piles being driven out of alignment. It is advisable to drive Z piles in pairs.

Figure 500 – 5: Steel Sheet Piling

A guide form or guide walers should be employed to result in well–driven, properly aligned sheeting. Sometimes the walers can be built in a movable trestle–like form. Distance between walers should be slightly wider than the back–to–back distance of the sheeting. To provide guiding stability for installing successive sheets, a wood wedge can be placed in the trough of the previously driven sheet, as shown is the following figure.

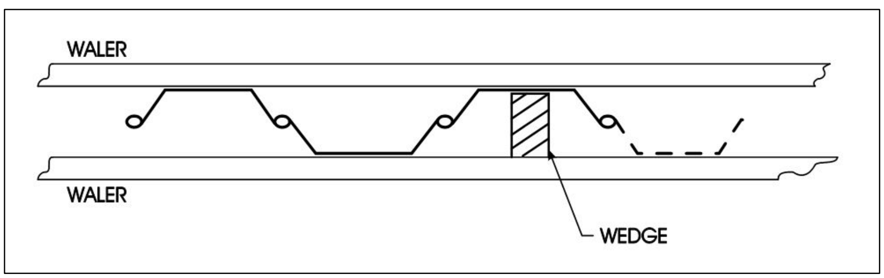


Figure 500 – 6: Guiding Sheet Piles with Walers and Wedge

D. Obstructions

If borings or other data show underground obstructions, the sheet piling should be driven in panels. When an obstacle is encountered, the driving operations should cease. The hammer should be moved to the next pile that can be driven, and with piles on both sides of the obstacle acting as guides, it often is possible to drive through the obstacle. Increasing the number of hammer blows may also be helpful in the operation.

E. Driving Tips

The following tips may assist in the drive piling operations:

- Plug the open interlock. Material is prevented from clogging the leading interlock in the first pile by forcing a bolt or similar object into the open case.
- Drive short sheet piles in soft ground to full depth singly or in pairs.
- To prevent creep in driving long piles or in hard ground, proceed as follows:
  - Set guide walers along the line of sheeting
  - Drive a pair of sheet piles to partial depth
  - Set a panel of a dozen single piles or pairs in the walers
  - Drive the last pile or pair in the panel part way
  - Drive the piles between the first and last piles or pairs to full depth
  - Drive the first pile to full depth
  - Drive the last pile two–thirds its full depth to act as a guide for the first pile of the next panel

Note: Good practice specifies that no sheet pile should be driven more than one–third of its length before adjacent sheet piling is driven.

- The piling should not be over–driven. Sometimes a driven pile may be drawn down by the next one being driven if the ground is very soft, or where high frictional forces develop in the interlock. To counteract this draw down, bolt the piles to a stiff wale, and if it happens before this precaution is taken, it usually is better to lengthen the drawn pile than to try to jack or lift it.
- If the sheeting is leaking along the interlock and needs to be further sealed to minimize water intrusion, clay cat litter, fine sand, rice, or *Speedy Dri* may be sprinkled over the leaking interlock. The sheet piling is then struck with a sledge hammer to enable the sealing material to migrate down the interlock and seal the leak.

F. Cofferdam Uplift

After the cofferdam has been constructed, the excavation has been completed, the required piles have been driven, and the soil reaction observed, the hydrostatic uplift should be calculated for the anticipated water level during the critical period that the cofferdam is to be dewatered. Since these values are assumed in the design, it is important that the actual dimensions are checked in the field during construction. Any variations should be brought to the attention of the Bridge Design Engineer and/or District Construction Engineer.

The example and graphs found at the end of this subsection are given as guidance for dewatering a cofferdam after the foundation seal has been poured. Using the minimum depth of the concrete seal, the graph can be used as a quick reference guide for determining the stability of the foundation seal. The height of water should be measured from the vent elevation of the cofferdam. Calculations should be performed if the variables plotted approach the dividing line.

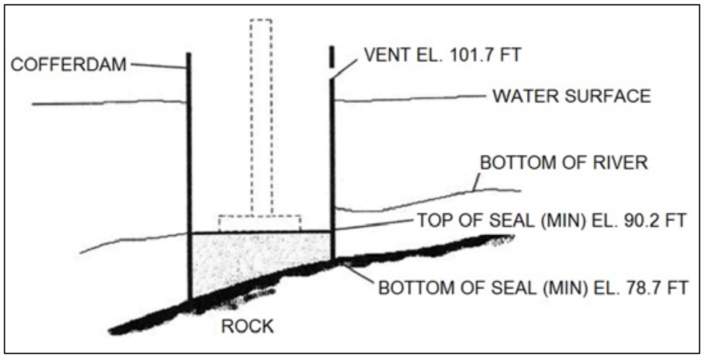


Figure 500 – 7: Measuring Cofferdam Uplift

When a foundation seal is to be poured on rock, the bottom of the cofferdam is to be inspected for total excavation and condition of the rock surface. This inspection is performed by divers under State contract. Contact the District Construction Engineer with sufficient lead time to arrange for this inspection when needed.

After the foundation seal has been poured on rock, an independent coring crew will be brought in to take core samples through the seal and at least one foot into the rock at each corner and one in the middle. These cores are inspected for voids, seams, or unsatisfactory concrete. These cores shall be labeled and placed in core boxes and remain on the project for future reference.

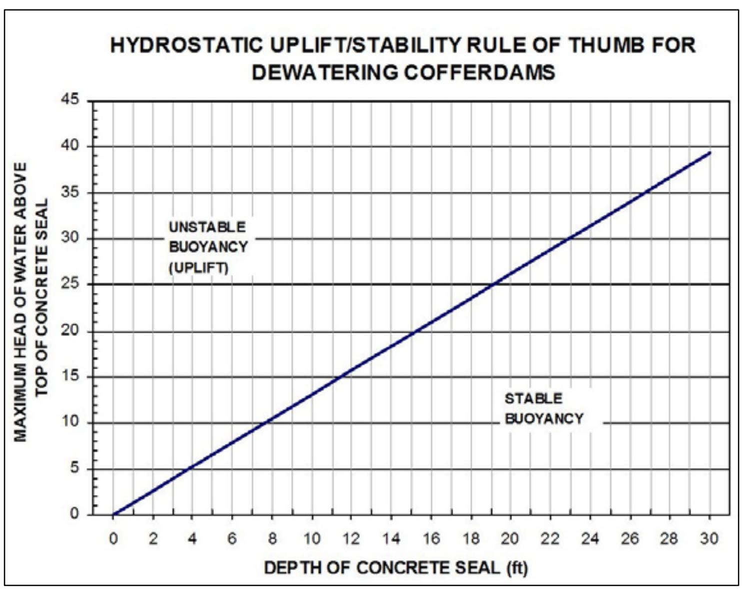


Figure 500 – 8: Cofferdam Uplift Graph (Max. Head vs. Seal Depth)

For this graph, the following are given parameters:

$ƛ_{CONC}$ = Unit Weight of Concrete = 144.2 lbs/ft^3,

and

$ƛ_{WATER}$ = Unit Weight of Water = 62.43 lbs/ft^3

The weight of the sheeting and the framing and the friction of the seal against the sheeting are not considered. The graph line balances the hydrostatic uplift pressure of the water with the weight of the concrete in the cofferdam. Any occurrences of unstable cofferdam buoyancy during dewatering operations must be brought to the attention of the Bridge Design Engineer and/or District Construction Engineer.

<u>Example</u>

Determine whether or not the cofferdam seal is stable under the following conditions:

Given (from graph):

H_S = Seal Height = 11.5 ft

H_{MAX} = Maximum Head of Water = 14.4 ft

Solve:

$H_{ACT} = EL_{VENT} - EL_{TOS}$

where

H_{ACT} = Actual Head of Water

EL_{VENT} = Vent Elevation

EL_{TOS} = Top of Seal Elevation

Therefore:

H_{ACT} = 101.7 – 90.2 = 11.5 ft

Actual Head of Water = 11.5 ft,

and

Maximum Head of Water = 14.4 ft

Since the actual head of water value is less than the maximum head of water value, the seal is stable.

Note: A higher vent elevation in this case, 106.6 ft or higher, would result in an unstable seal.

SECTION 504 – BRIDGE EXCAVATION

504.1 – GENERAL

This item involves the excavation of earth and rock material for the construction of bridge substructures, footings and seals. The Contract Administrator should contact the Bureau of Highway Design's survey section for initial bridge control layout.

504.3 – CONSTRUCTION OPERATIONS

Common Bridge Excavation is a final pay (F) quantity item. This means that the quantity does not need to be measured, but shall be paid as the estimated bid quantity. However, enough information should be gathered before any excavation begins so that a quantity could be calculated if it appears that the actual quantity is going to differ with the estimated bid quantity by more than 25%. Before any excavation operation is begun, the Contract Administrator should take sufficient original cross–sections to allow excavation quantities to be computed as simply and as accurately as possible.

Cross sections for small structures often can be laid out by using centerline of bearing, channel, or roadway as a base line. Care should be taken to obtain cross sections at points of zero excavation and points of full width excavation as shown in the following figure.

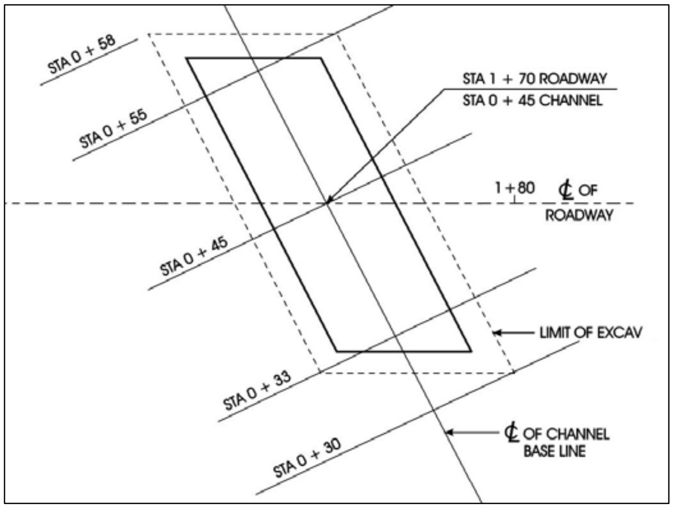


Figure 500 – 9: Bridge Excavation Survey

Base lines, stations of cross–sections, and limits of excavation should be plotted on the record plans. In most cases this data will be for information only. However, if it becomes clear that there is a gross error (greater than 25%) between the actual and estimated bid quantities, then this information must be plotted and used to calculate the actual quantity of common bridge excavation.

When possible, the base line should be laid out along the principal axes of the structure in order to reduce the number of intermediate sections. More than one base line can be used when the additional base line would simplify the cross–sections.

After cross–sections have been taken, control points should be set by the Contractor to provide ready reference to lines and grades. The Contract Administrator should verify the control point locations after they have been established, as unnecessary errors have often resulted from the movement of poorly set control points.

The Contract Administrator should not permit large unsupported holes to be dug if nearby buildings, utilities, bridge structure units, or sloping ground surfaces may be affected. It may be necessary to modify adjacent slopes or support the sides of the excavation to protect adjacent structures and provide a safe working area. Often, sheet piling is driven to protect adjacent structures, and, if necessary, the sheets can be cut to elevation and left in place if needed. The District Construction Engineer should be consulted if leaving the pilings in place is anticipated.

During excavation operations, the Contract Administrator should inspect the material carefully at successive levels to ensure that it corresponds with the boring logs. Soil conditions may vary considerably in a given area. Should the material differ substantially, the Contract Administrator should determine whether the bearing capacity is adequate. If in doubt, the Contract Administrator should contact the District Construction Engineer and the Soils Engineer at the Bureau of Materials and Research. Refer to safety requirements in the contract for shoring and/or sheeting requirements.

In conjunction with Item 503, the underwater inspection of completed excavation to rock surface condition is conducted by independent divers. When this inspection is needed, the Contract Administrator should contact the District Construction Engineer.

The following figure depicts an abutment with a skewed, elongated, stepped wing wall, and a flying wing end that uses an additional base line and match line.

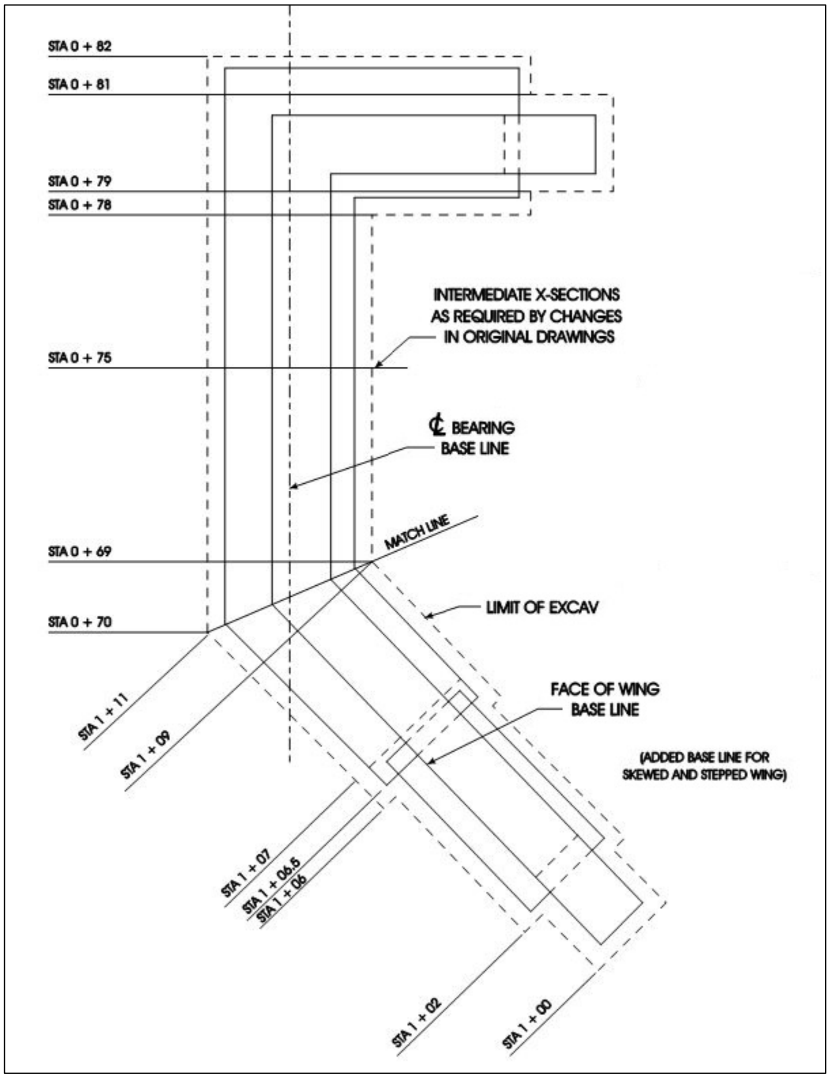


Figure 500 – 10: Bridge Excavation Survey Using an Additional Baseline

SECTION 506 – SHEET PILING

506.1 – GENERAL

Sheet piling is used to prevent scouring under structures, to prevent shore line erosion from moving water, and to provide a bulkhead. Sheet piling ranges from simple wood planks and light gauge sheet metals to heavy sections made of structural steel members.

506.3 – CONSTRUCTION OPERATIONS

A. Layout

The Contract Administrator should inspect the layout of the line along which the piling will be driven. Once the Contract Administrator is satisfied with the layout, an appropriate location for an offset line, to be used as a control line for the work, should be determined where it will not interfere with equipment performing the driving operation. Location of the control line should be discussed with the Contractor's superintendent, and once it is set, it the Contractor is responsible for ensuring that this agreed upon location is maintained.

B. Measuring

In order to measure the cutoffs, paint marks should be placed approximately 3 ft from the top of each pile before driving. Then the in–place length and cut–off lengths can be computed and recorded. Marking can usually be done well ahead of driving so that the Inspector is free to keep track of the driving.

SECTION 508 – STRUCTURAL FILL

508.3 – CONSTRUCTION OPERATIONS

The plans should be closely followed relative to the minimum depth of excavation and the dimensions to which the structural fill is to be placed. Follow the notes on the plans or as stated in any addendums relative to placement of adjacent embankments in conjunction with the structural fill item.

Preparation of the area for structural fill is accomplished under bridge excavation. Refer to *Section 504 Bridge Excavation* for more information. Generally, crushed gravel is the material used for Item 508 unless otherwise specified. If 1½ in angular stone is used the maximum depth allowed shall be discussed with the District Construction Engineer and the Soils Engineer at the Bureau of Materials and Research. The Contractor must place stone in appropriate lifts and needs to use at a minimum, a mid–weight plate compactor to properly consolidate the stone to reduce voids and to maximize stone interlock. At no time should stone be placed to grade and leveled off without compactive effort or differential settlement is bound to occur.

After the structural fill is in place, the Contractor must keep the structural fill dewatered to maintain the maximum density until the footing concrete has been placed. To accomplish this, a simple ditch around the perimeter of the structural fill can be made to feed into a sump pump located at one corner of the excavation outside the limits of the footing and formwork.

During winter construction, the Contractor must protect the structural fill from freezing before and after the concrete has been placed and until it is properly backfilled.

SECTION 510 – BEARING PILES

510.1 – GENERAL

The pile, as a structural member, is used to transmit the load of a structure through a fluid or stratum of low bearing value to one of more adequate capacity; provide stable foundations in areas subject to scouring action; consolidate loose granular soils to some degree by driving high volume wedging action piles; anchor structures against uplift or overturning; and act as protective devices on piers in the form of fendering.

Bearing piles function in three ways:

- The end bearing pile acts as a supporting member to transmit a load through semi–fluid or soft strata to hard material or rock.
- A friction pile transmits the load to the soil throughout its entire length by the friction of the soil against the pile.
- The compaction pile is intended to compact relatively loose soil by high volume wedging action.

Note: Certain conditions may call for the use of a combined end bearing and friction pile, which relies on transmission of loads with the bearing characteristics of both types of piles.

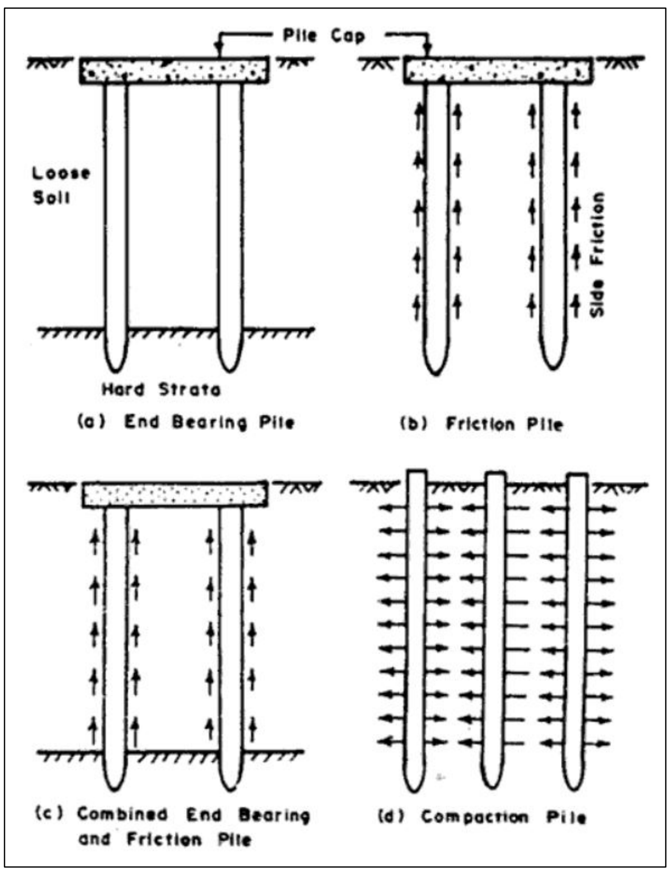


Figure 500 – 11: Bearing Pile Types

In the design stage, consideration should be given to the use of batter piles when unstable soils are encountered. If vertical piles go down through an unstable soil and then come to refusal with only comparatively small penetration into the firm material, they have very little lateral stability. The weight of the approach embankment behind the abutment may be sufficient to cause the abutment either to tilt or to move bodily toward the stream. The unstable material flows under the pressure of the fill, moving the piles with it.

The amount of movement will depend upon the soil condition and length of piles, but cannot be predetermined. When a critical soil conditions are encountered, the Contract Administrator should discuss the need for any possible changes to the plan pile configuration and design criteria with the District Construction Engineer.

510.2 – MATERIALS

A. Timber Piles

Timber piles, though rarely used, shall conform to ASTM *D 25, Standard Specification for Round Timber Piles*, Class B, clean–peeled, and shall be preservative treated.

B. Precast Concrete Piles

Precast concrete piles are steel reinforced members that are cast and thoroughly cured before driving. Precast piles include conventionally reinforced piles and prestressed concrete piles. Both types are available in a number of different cross sections. Prestressed piles are more prevalent due to their greater resistance to damage during driving.

In driving precast concrete piles, care must be taken to cushion the top of the pile from the direct impact of the hammer blow. Pile cushions for concrete piles should have the required thickness determined from a wave equation analysis but should not be less than 4 in. A new plywood, hardwood, or composite wood pile cushion that is not water soaked should be used for every pile. The cushion material should be checked periodically for damage and replaced before excessive compression (more than half the original thickness), burning, or charring occurs. Wood cushions may take only 1,000 to 2,000 blows before they deteriorate.

During hard driving, more than one cushion may be necessary for a single pile. The cushion must give enough protection to prevent local damage to the pile, without absorbing too much of the energy of the blow. Indifferent handling of piles may cause incipient cracks to form. These cracks may open up during driving or may even spall and “powder” to such an extent as to seriously lessen the strength and life of the pile.

Precast piles should not be shipped from the plant or driven until they have obtained sufficient concrete strength to withstand handling and driving stresses. If the precast piles have been allowed to dry after curing, they should be wetted at least 6 hours and kept moist before being driven.

Figure 500 – 12: Driving Precast Concrete Piles

C. Cast–in–Place Concrete Piles

Cast–in–place concrete piles are made by forming a hole in the soil and filling it with concrete. There are two general types of cast–in–place concrete piles:

Shell: Either a steel shell, which is heavy enough to be driven without a mandrel, is used, or a light steel shell is driven with a mandrel, which is later removed. The shells for piles cast–in–place shall be carefully checked after driving for water tightness and deformation due to the driving of adjacent piles. A mirror for reflecting sunlight into the shell is the most common method for this check. On cloudy days, a flashlight may be lowered into the shell to aid inspection.

Any water contained in pile shells should be siphoned out immediately prior to placing the reinforcing cage and concrete. If shells are not going to be filled immediately they should be covered and re–examined again prior to the placement of concrete.

Shell–less: The shell–less type of pile is made by driving a light steel shell into position with a mandrel or an earth auger, then filling it with concrete.

Figure 500 – 13: Pouring a Cast–in–place Concrete Pile

D. Steel Piles

Steel H–piles are the most common piles in use in New Hampshire due to the general soil conditions. Of all the materials used for foundation piles, steel is the only material that has an ultimate strength in compression comparable to that of hard rock. Therefore, H–piles are used effectively in point bearing applications. In certain types of soil conditions steel piles may also rely on the "skin friction" between the pile and the soil to support a portion of the load. Steel pipe piles are used for applications where lateral strength is required in all directions. Closed–end piles are often driven to refusal and subsequently filled with concrete.

Figure 500 – 14: Steel H–Piles

510.3 – CONSTRUCTION OPERATIONS

The Contract Administrator has many duties to perform during pile driving operations, and consequently should become thoroughly familiar with the plans and Specifications. The

Contractor shall submit a plan depicting the type of driving equipment, energy ratings of the hammer, and proposed lengths and sizes of the piles to the Bureau of Materials and Research Soils Engineer through the Bureau of Construction so that a minimum blow count can be calculated using the wave equation formula. This blow count will be given to the Contract Administrator as a requirement to be met to ensure the load capacity has been achieved without overdriving and possibly damaging the pile.

A dynamic pile test is often included in the contract as a special provision. When called for, personnel from the Bureau of Materials and Research will be on site to conduct the dynamic pile test. A representative pile will be selected and equipped with instrumentation to evaluate driving stresses. This information will be used to adjust or verify the wave equation information.

A. Driving Piles (General)

The following general conditions and parameters regarding penetration, bearing, and location apply when driving piles.

- Penetration Requirements: Plans usually show an estimated tip elevation or the notation that piles shall be driven to refusal on bedrock. Action of a given pile section being driven by a specific hammer into bedrock material of varying degrees of hardness or decomposition will have to be evaluated at the work site. The borings should be studied carefully to determine the nature and characteristics of the bedrock; the nature of the overburden above the bedrock; and the reason(s) for specifying practical refusal instead of a specific bearing value.

- Bearing Requirements: For most projects, piles will be driven to develop a computed bearing of not less than the design load stated on the plans. Plans and Specifications for large bridges often require load tests on various piles. The main purpose of a load test is to verify results indicated by the bearing formula. The first load test should be made early in the pile driving operation before any piles have been cut off.

- Location Requirements: The Contractor should lay out the individual pile locations and NHDOT project personnel should check them for conformance with the plans. The pile layout is usually completed by referencing the roadway centerline and centerline of bearing as well as established control points and then specifying each pile location with string lines and tape.

  All piles, when driven, must be in line and in their true position. The Contract Administrator should make every attempt to minimize deviations to the planned layout and in no case should deviations exceed tolerances allowed in the Specifications. Any questions regarding tolerances should be directed to the Bridge Design Section.

B. Preparation of Piles for Driving

When piles arrive on the project, they should be inspected by the Contract Administrator. In the case of epoxy coated piling, it may be necessary to touch–up scratched areas. Timber piles should be checked thoroughly for compliance with Specifications. During inspection, piles should be marked every 1 ft for blow count determination, thus preparing them to be placed in the leads for driving.

Timber piles are susceptible to damage during driving, particularly under hard driving conditions. To protect the pile against damage, the following procedures should be followed.

Cut the butt of the pile off square and chamfer it so that the hammer strikes evenly on the butt. The chamfered butt must fit the driving cap or hammer base, where used. If crushing, brooming, or splitting occurs, in addition to chamfering the butt, wrap the top end of the pile with 10 or 12 turns of heavy wire at a distance of about one diameter below the head of the pile. An alternative to wire wrapping is to clamp two half–rings of ⅜ in steel around the butt and then bolt them together. If a hole is bored in the butt of the pile, double wrappings should be used.

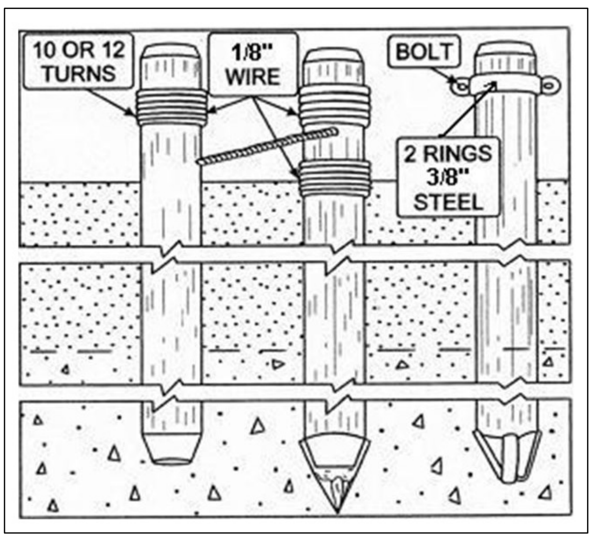


Figure 500 – 15: Methods for Protecting Timber Piles

The pile is usually pointed by sharpening the tip to the shape of a truncated pyramid; the blunt end may be 4 to 6 in square. The length of the point may be from one–and–a–half to two times the diameter of the tip. In extremely hard driving, steel points or shoes may be used to protect the tips of the piles. Shoes of various sizes are available from manufacturers of pile driving equipment. Tips should only be employed when specified on the plans.

Steel H or pipe piles are driven with caps specially designed to fit over the tops of the piles. The points of steel H–piles may be reinforced by adding welded or riveted plates**.** The thickness of the web and flange may be built up to two–and–a–half to three times the original for a height of two–and–a–half to three times the width. At the mill, or in the field, the lower ends of steel pipe piles may be fitted with flat plate ends, pressed steel points, or special cast or fabricated steel shoes for open–end driving.

Pile Design lengths should be checked against the plan pile notes and borings prior to the Contractor ordering them to avoid unnecessary splices or large quantities of cut offs. Wherever possible, steel piles should be ordered from the mill cut to the required length, in order to avoid field splicing or cutting.

Any holes burned through the piles for lifting purposes must be plug welded if their location occurs below the bottom elevation of the concrete foundation. As an alternative, holes may be burned through the neutral axis of the pile; this case do not require plugging.

C. Driving Procedure

The procedure for driving piles consists of four basic steps.

1. The pile driver is brought into position with the hammer and cap at the top of the leads.

1. The pile line is lashed to the top of the pile and then the pile is raised in the leads. Next, the tip of the pile is placed in the proper position.

2. The pile is centered under the pile cap and the pile cap and hammer are lowered to the top of the pile. If a drop hammer is being used, the cap is unhooked from the hammer.

3. In the case of a drop hammer, the hammer is then raised and dropped to drive the pile. The driving is started slowly with a drop hammer, raising the hammer only 2 to 4 in for each blow until the pile is firmly set. The height of the fall may then be increased to a maximum of 10 ft or so. Blows should be applied as rapidly as possible, in order to keep the pile moving.

With steam or pneumatic hammers, operating pressures should be restricted until the pile is firmly set. The pressure may then be increased to the recommended value. For diesel pile hammers, the recommendations of the manufacturer should be closely followed.

D. Positioning the Guiding Piles

When piles are driven on land, and where a reasonably liberal tolerance is permitted as to trueness and position of the top of the pile, the position of each pile may be marked with a stake. In driving steel H–piles by this method with suitable leads used, the final position of the top of the pile should be within about 2 in of its theoretical position, and it should be plumb within about 1 in per 5 ft of length. It is generally desirable to systematically offset positioning stakes from the final desired pile position, so that the stake will remain in place as the tip of the pile is maneuvered into position.

A single guide frame may be constructed to hold the piles in position during driving where closer tolerances are required. Occasionally, it is required that piles be driven with very close tolerances. In such cases, it is necessary to erect an accurate guide frame at or close to the ground (or water) level with a second frame 20 to 30 ft or more above it, depending upon the length of the piles and the distance they extend above ground or water level. In this manner the piles are held plumb and exactly in position. Driving accuracy within ¼ in of the exact position is possible by this method.

Whenever possible it is desirable to drive the vertical piles first, often located in the back row of the pile location plan of the footing. After these are driven, the guide frame, or template as it is more commonly referred to, can be placed. Usually a pile is laid on the ground along the line of battered piles. After it is properly located it is then braced back to the previously driven vertical piles. This template facilitates the placement and driving of the battered piles.

When piles are to be driven in water, one of several methods may be used to mark the desired pile positions. When a number of bents are to be constructed, a stake is placed at each abutment approximately 6 in from the pile centerline. A wire rope is stretched between the two stakes and a piece of tape or cable clip is fastened to the rope at each pile position. When a floating pile driver is used, a frame for positioning piles may be fastened to the hull.

Figure 500 – 16: Timber Pilings in Water

E. Precautions to Observe During Driving

Watchful monitoring of pile driving operations should ensure that no damage to the pile or pile hammer occurs. The following precautions should be observed:

- The pile driver must be securely braked or fastened down to prevent movement during driving.
- The hammer hoist line must be kept slack at all times while the pile is being driven, so that the full weight of the hammer rests squarely on the pile. It is essential that the fall of the hammer be in line with the pile axis; otherwise, the head of the pile may be damaged severely, the hammer damaged, and much of the energy of the hammer blow lost.
- If the hammer bounces, it may be too light, but this usually occurs when the pile has met an obstruction or the pile has penetrated to a solid layer. When a double–acting hammer is being used, hammer bouncing may be due to the use of too much steam or air pressure.
- When the last few blows of the hammer will not drive the pile more than the computed penetration per blow for the required bearing value of the pile, penetration has stopped because of an obstruction or refusal. This penetration applies only when the driving is being done with the right size hammer operating at rated energy.

Note: Further driving at this point will inevitably damage or "cripple" the pile.

If the lack of penetration seems to be due to an obstruction, it may be small enough that 10 or 15 blows of less than maximum impact will drive through the obstacle and start the pile moving again. If the pile has encountered a firm stratum, this fact may be detected by driving a few other piles nearby.

F. Driving Bearing Piles in Groups

When piles must be driven in closely spaced groups, the following conditions and parameters should be observed:

- In a sand or gravel deposit, the soil must be compacted or displaced an amount equal to the volume of the pile. If the deposit is quite loose, the vibration of pile driving frequently results in considerable compaction of the soil. The surface of the ground between piles then may subside or "shrink." Careful watch must be kept to ensure that this subsidence does not cause any damage to the foundations of nearby structures.
- If piles are driven into dense sand and gravel deposits, some ground heaving may occur. Clay soils are relatively incompressible under the action of pile driving. Hence, a volume of soil equal to that of the pile usually will be displaced. This results in ground heaving between and around the piles.
- The driving of a pile alongside those previously driven frequently will cause those already in place to heave upward. Such cases may be detected by taking level

readings on the tops of piles previously placed. Piles that heave greater than ½ in must be re–driven to firm bearing.

- Displacement of soil by the pile may create sufficient lateral force to move previously driven piles out of line. Serious damage may result to shells driven in the construction of cast–in–place concrete piles or to "green" cast–in–place piles of the shell–less type.

The sequence of driving piles in groups should be as follows:

- Driving should progress from an area of high resistance to low resistance toward a stream, or down slope. This minimizes shoving previously driven piles out of place when succeeding piles are driven.
- Outer rows in the group should be driven first if the piles derive their principal support from friction. Inner rows are driven first if the piles derive their support from point bearing.

G. Battered Piles

The presence of battered piles dictates the sequence of driving all piles. The Contractor must decide the driving order so that the battered piles can be driven at the correct angle and location without creating obstacles for the remainder of the operation. The driving area, particularly if it is structural fill, must accommodate the driving equipment and crane. The intended batter angle must be rigidly held.

H. Pile Driving Inspection

The Inspector should continuously observe the driving operations, and at the same time, relate the rate of movement of the pile under the driving force to the exploratory data provided on the plans. The Inspector should not make the error of allowing driving to be terminated at an elevation above the "minimum penetration" elevation when the tip of the pile has hit a thin, hard layer that may overlay a softer material. When driving piling of any type, the Inspector shall keep the Contract Administrator advised of any unusual changes in driving performance.

Pile driving operations should be continuous whenever possible. Interruptions of driving will falsely show on the driving record as an indication that the pile is nearing refusal. Whenever pile driving is interrupted, the driving record for that pile should so indicate, so that the data will not be misinterpreted as indicating a hard driving spot for this pile.

Upon the continuation of driving, the pile may not move even under maximum stroke due to friction forces that have "grabbed a hold" of the pile while it's been sitting in its partially driven state, particularly if the driving operations don't resume until the following day.

The effort must be continued, however, because the pile will break loose and continue to penetrate.

Extra care must be taken to make sure that the hammer starts off with a smaller stroke on such a pile. The friction forces that have built up will resist the initial stroke of the hammer causing a higher stroke rebound than desired. This increased load due to the higher hammer stroke could very easily cripple the pile.

I. Driving Through Obstructions

Frequently, obstructions are encountered below the surface of the ground during pile driving operations, particularly when piles are driven in the industrial and commercial areas of older cities. They are a matter of considerable concern since they may prevent a pile from penetrating far enough to provide adequate load–carrying capacity.

When an obstruction like a rotten log or timber is encountered, 10 to 15 blows of the hammer may cause the pile to break through it. Another method would be withdrawing the pile and configuring its leading edges to a chisel shape to further aid in cutting through obstructions. If the obstruction cannot be breached by these methods, a small explosive charge, lowered to the bottom of the hole, may be necessary to blast it out of the way.

J. Underwater Pile Driving

It is sometimes desirable to drive piles underwater, rather than by the use of a pile follower. Both double–acting and single–acting air or steam hammers are suitable for driving underwater when properly handled and rigged. The recommendations of the hammer manufacturer in preparing and rigging the hammer should be followed. Special attention must be given to proper lubrication of the hammer.

K. Effects of Driving on Adjacent Structures

When piles are to be driven for a new foundation alongside an existing structure, precautions must be taken to ensure that the existing structure is not damaged. Shrinkage or heaving of the ground around the new piles may be responsible for serious damage.

In such cases, careful records on the levels of ground in the area and the behavior of adjacent foundations must be kept. If piles are to be driven behind a retaining wall, the pressure on the wall may be greatly increased due to consolidation of a granular soil by vibration, while a plastic soil may actually be forced against the wall. Also, if battered piles are to be driven within a cofferdam, the cofferdam should be installed at a sufficient width to avoid conflicts with the piles as they are driven outward underground.

In critical locations, special methods of placing piles – such as jetting or jacking – may have to be used.

L. Straightening Piles

Piles should be carefully examined during driving for proper orientation and alignment. This is particularly important for the initial 10 to 20 ft of penetration. Pile driving should be immediately terminated if any significant twisting or bending of the pile is observed. In such a case, the cause of the problem should be evaluated. Removal and relocation of the pile or pre–boring may be necessary.

In general, procedures to twist or force a pile into proper alignment should be avoided since this can cause overstress in the pile. Piles should be straightened as soon as any misalignment is noticed during driving to meet job Specifications. The greater the penetration along the wrong line, the more difficult it is to get the pile back into plumb.

The following are commonly used methods of realigning a pile:

- The use of a block and tackle, with the impact of the hammer jarring the pile back into line.

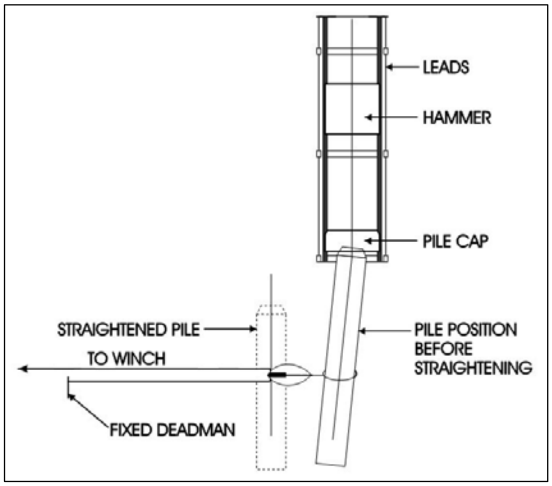


Figure 500 – 17: Straightening a Timber Pile with Block and Tackle

- The use of a jet may be used in combination with the block and tackle method.
- The use of an alignment or straightening frame constructed of timber and bolts to force improperly driven and aligned timber piles back into alignment.

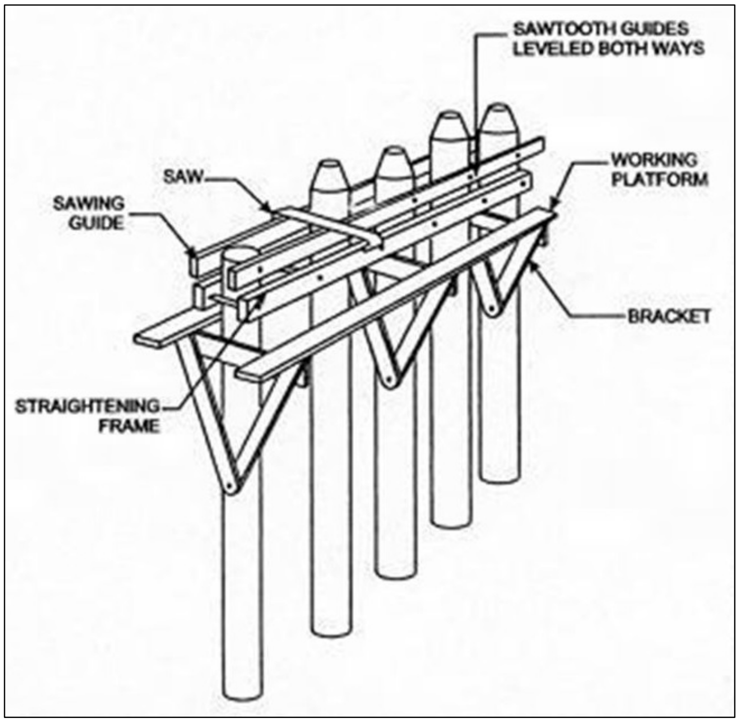


Figure 500 – 18: Using a Straightening Frame to Properly Align Timber Piles

- The use of a vibratory pile driver to partially extract and realign the pile.
- Realignment of steel H–piles frequently must include twisting of the individual piles to bring the webs of the piles into the proper position.

M. Length Adjustment

For timber piles, length adjustment is a relatively simple matter. Normally, after driving, the butts of piles are 2 to 3 ft above the desired finished elevation. The piles are then sawed off at the desired level. For timber piles in a bent, a good method is to nail sawing guides across all piles in the bent.

Timber piles that are too short are easily spliced. The splice must be strong and stiff in order to develop the necessary resistance to bending. Sleeve pipe joints may be used for splicing and usually are made of 8 to 10 in diameter steel pipe cut in 3 ft lengths. Contact ends must be carefully cut to give full contact, and pile ends must be trimmed to fit snugly in the pipe. A flat transverse bar through the sleeve between the abutting pile ends will keep the sleeve in place during driving. Bolting timber or steel splice plates may also be used.

Steel piles are adjustable in length by cutting or splicing. Splices can be made by bolting, riveting, or welding. When welding two sections together they must be butt–welded. It is customary to order piles that are to be spliced cut to mill tolerances. Milled ends are unnecessary since scarfing of the piles in preparation of butt–welding is done in the field by torch–cutting.

Shells for cast–in–place concrete piles are easily adjusted in length by cutting or welding. Adjustment in length of precast concrete piles is somewhat more difficult. The Special Provisions or the Bureau of Bridge Design should be consulted for details of the procedure.

N. Welder Qualifications

Welders shall meet the qualifications listed under *Subsection 550.3.6.2 Qualification of Welders and Welding Operators* of the Standard Specifications. Welders responsible for welding pile splices shall be groove or "G" qualified. Fillet "F" welds are not sufficient when welding pile splices. Refer to *Subsection 550.3(F)* for more information.

O. Pile Driving in Cold Weather

It is possible to conduct pile driving operations in severe cold weather conditions, even though the ground is frozen. Frost up to 2 ft in thickness can be broken through successfully by driving a heavy pilot pile or a heavy casing. Another way to "break" frost is to thaw the ground by spreading unslaked lime over the area to a depth of 3 to 4 in, covering the lime with snow, covering the snow with tarpaulin, and then covering the tarpaulin with snow again. This method may melt through a layer of frost 3 ft thick within about 12 hours.

Earth augers are capable of drilling holes in frozen ground and may be used to aid pile driving. The driving of piles for bridge foundations may be somewhat easier during cold weather, in that it may be possible to work through sound river ice. Holes cut through the ice may act as guides for the piles.

Special instructions of the manufacturer concerning operation of steam, air, or diesel pile hammers during cold weather should be followed. Contractors will often spray ether into the ports of a cold diesel hammer to help get it started

P. Avoiding Damage to Concrete Masonry Due to Pile Driving

The Contractor should coordinate its pile driving and concrete placement operations in such a manner that no damage or displacement will occur to concrete masonry in any existing substructure unit as a result of pile driving operations in any other unit.

To the extent practicable, all pile driving within a substructure unit should be completed before any concrete is placed in that unit. Should it become necessary to drive piling or conduct other construction operations that might adversely affect freshly placed concrete, including blasting and demolition of existing structures, consult with the District Construction Engineer. A period of 72 hours is often required between placing concrete and starting or resuming nearby pile driving operations.

Q. Pulling Piles

It is frequently necessary to pull piles from the ground in order to reuse them. This is particularly true of steel sheet piles used in cofferdams, temporary retaining walls, and so on. The most effective way of pulling a pile is by using a vibratory pile extractor while exerting a strong upward pull. Commonly, specially designed double–acting extractors are used, but it is possible to rig a pile hammer in an inverted position to pull piles.

R. Pile Driving Equipment

Pile driving equipment in current use on NHDOT projects includes six types of pile driving hammers:

- Drop Hammers
- Single–Acting Hammers
- Double–Acting Hammers
- Differential–Acting Hammers
- Diesel Hammers
- Vibratory Hammers

Single–acting, double–acting, and differential–acting pile hammers depend upon steam or compressed air for motive power. A general term for these is steam or pneumatic hammers. There are two types of diesel hammers: open–end and closed–end. One other type of hammer, which is in limited use, is the sonic hammer.

The Contractor will provide the Contract Administrator with the manufacturer, model, and type of each hammer that is planned for use on the project. The Contract Administrator requires relevant data for each hammer, including the hammer's energy and power values, ram weight, and stroke distance, for proper pile driving calculations.

The Materials and Research Soils Engineer or the Bridge Design Engineer must approve the use of each hammer on the project prior to commencing any pile driving operations.

Pile Dynamics, Inc., a Cleveland, OH company that produces foundation dynamic testing equipment, has a list on its website of pile driving hammer parameters based on the wave equation analysis software program, GRLWEAP. This *GRLWEAP Hammer Database File* may be found at the following URL:

http://www.pile.com/pdi/products/grlweap/hammers.aspx

Manufacturers of air, steam, or diesel type driving hammers designate each size of each type of their hammers by a number and the rated energy output of the hammer. This designated energy rating is usually the maximum energy that the hammer is capable of producing as determined by the manufacturer by use of a formula or actual energy measurement.

The operating speed of mechanical hammers is also critically important. The speed must be maintained to produce the energy intended.

The following is a list of the various types of pile driving hammers and their operating characteristics.

- Drop Hammers

  A drop hammer basically is a block of metal that falls on the end of the pile. Standard drop hammers are made in various sizes and weights. For example, Vulcan Iron Works of Chattanooga, TN, makes several models that range in weight from 500 to 3,000 lbs. Hammers are fitted with a round steel pin for use in hoisting. A triangular die, grippers, and trip devices may be supplied if required.

  Similarly, the Eagle Iron Works, Des Moines, IA, manufactures several standard drop hammers which range in weight from 1,500 to 7,000 lbs. Hammers and accompanying follow blocks are made of tough semi–steel. Hammers are supplied with leaded–in steel pegs for fastening the follow block sling.

  The distance through which the hammer falls can be varied within the limits of equipment available. The drop hammer is the simplest type of pile hammer. However, it is comparatively slow in comparison with other types.

  When a particular hammer manufacturer's Specifications are unavailable, drop hammers must be weighed before any piles are driven. The drop hammer stroke should be carefully measured. This can be done by taping a piece of rope or rag around the hammer line at the height above the hammer for the drop desired. The operator can then gauge the drop with reasonable accuracy. Short fast strokes of the hammer are more efficient than long slow strokes.

  - Single–Acting Hammers

  In general terms, a steam or air hammer consists of a stationary cylinder and a moving part (ram), which includes the piston and striking head. A single–acting hammer is designed so that the ram is raised by steam or air pressure and drops by gravity.

  This type of hammer is marked by a heavy ram, low impact velocity, and a comparatively small number of blows per unit of time. Single–acting hammers are most powerful of this type of hammer, and they come in a wide variety. For example, Vulcan supplies five standard models of this type. Typical of Vulcan hammers is Model No. 2, which has a 3,000 lb ram, a rated striking energy of 7,225 ft–lbs per blow, at a rated speed of 70 blows per minute.

The McKiernan–Terry Corporation of Dover, NJ, also manufactures several hammers of this type, including Model S–20, which has a 20,000 lb ram, a total striking energy of 60,000 ft–lbs per blow, and a rated speed of 60 blows per minute.

Because of its operating characteristics, this type of hammer is generally used where heavy piles; piles of low compressive strength (such as precast concrete); or piles are to be driven in stiff clay, compact gravel, or other types of dense soils.

- Double–Acting Hammers

  The piston of a double–acting pneumatic or steam hammer is raised by the motive fluid and, in addition to the gravity fall, the piston is forced down by pressure. Double–acting hammers are characterized by high frequency of blows, usually from 90 to 100 blows per minute. They are extensively used for driving sheet piles and bearing piles through common soils. The high frequency of blows results in faster driving by keeping the pile moving and preventing the buildup of soil friction.

  A disadvantage of this type of hammer is the relatively high impact velocity, which may result in excessive deformation of the head of any low compressive strength pile.

  A variety of sizes of double–acting hammers are available from manufacturers. Models range from extremely light hammers with rams that weigh less than 100 lbs up to heavier units. One large hammer of this type – the McKiernan–Terry Model 11B3 – has a ram that weighs 5,000 lbs, an 18 in stroke, and an operating frequency of 95 blows per minute.

- Differential–Acting Hammers

  A differential–acting pile hammer is a variation of the double–acting pile hammer. The piston and cylinder are designed to reduce the size of prime mover required. Ram weights are greater than for comparable double–acting hammers in order to obtain lower impact velocity. Frequency of blows is greater than for comparable single–acting hammers.

  Vulcan Iron Works manufactures seven different models of open–type, differential–acting pile hammers of different sizes and weights. Typical of these is Model 80C. Striking parts for this hammer weigh 8,000 lbs, the rated striking energy per blow is 24.48 kip–ft/kW, and the frequency of blows is 111 per minute. Vulcan's Model DGH Portable Pile Hammer is a small differential–acting hammer that has a gross weight of 850 lbs.

- Diesel Hammers

Open–ended Diesel Hammer: This type of hammer has an open upper end, whereby the ram is unrestricted in its rebound and is visible above the body of the hammer. The height of the rebound is taken as the length of the stroke for the following blow. Under normal driving conditions, the height of rebound will increase as the resistance of the pile to driving increases.

There is a force exerted on the pile by the explosion of the charge of Diesel fuel, and likewise there is a loss in the kinetic energy during the fall of the ram due to the cushioning effect the explosion of the fuel has on the impact of the ram. It is assumed that the energy gain of one is about equal to the energy loss of the other, and therefore the energy output for this type of hammer in foot–pounds is the product of the weight of the ram and the length of stroke.

Measurement of the length of stroke is made by observation, reading on a graduated rod attached to and extending above the hammer body or shell, the height to which the top of the ram reaches in its rebound.

It is important to note that blow counts determined by the wave equation analysis method apply only when the hammer is operating at the proper (often maximum) stroke. The Inspector will develop a sense of full desired stroke based on visual obstruction of the cylinder, the tone of the hammer's ring, and the time interval between blows.

Closed–ended Diesel Hammer: This type of hammer has a ram that operates in a cylinder that is closed at the top and the upstroke of the ram traps and compresses air in the bounce chamber, which is the space in the cylinder above the top of the ram. The energy stored in the compressed air is imparted to the ram on the downward stroke. The output energy of the hammer is designated the equivalent energy measured in foot–pounds. This equivalent energy consists of the product of the weight of the ram and the length of its stroke; and the force due to the compressed air in the bounce chamber, which in turn is equivalent to an additional height in the fall of the hammer.

As the resistance of the pile increases, the force of explosion of the Diesel fuel, acting on the ram in its upstroke, increases, and the increased energy of the ram increases the energy stored in the compressed air chamber, which in turn provides an increase in the force imparted to the ram at the start of its downstroke. Thus, it is evident that the energy output of this hammer, within the limits of its rated energy output, will increase as the resistance of the pile being driven increases.

When determining the bearing capacity of a driven pile, it is necessary, at the time of the count of the blows of the hammer and measurement of set of pile, to determine the equivalent energy of the hammer. From a gauge attached to an air hose that is connected to the bounce chamber, readings of the air pressure are made and these pressure values are converted to equivalent energy in joules by means of charts prepared for the hammer and furnished with the gauge.

In the event the resistance of the pile during the driving is sufficient to produce an air pressure in the bounce chamber which, as read by the gauge and converted by chart into equivalent energy, is of a value greater than the manufacturer's rated energy for the hammer, the manufacturer's rated energy for the hammer should be used in the bearing formula.

- Sonic Hammers

The sonic hammer has the advantage of operating with almost total lack of noise and vibration, which is most desirable in urban areas. However, the rate of penetration varies little with changes in soil conditions and the speed of penetration makes it difficult to determine the bearing capacity of friction piles. Therefore, these hammers lend themselves best to sheet–pile and end–bearing pile applications until some means of determining the accurate driving energy is developed.

Sonic and vibratory hammers should not be used for the driving items without prior approval of the Contract Administrator and the Bureau of Materials and Research. Load–bearing piles driven with these hammers must be "tested" through the use of another type of hammer for which blow count energy can be determined.

- Vibratory Hammers

Vibratory pile driving hammers, powered by hydraulic motors, are configured with counter–rotating eccentric weights. This kind of hammer optimizes the direct application of vertical vibrations into the pile while canceling out horizontal vibrations.

Vibratory hammers are used to both drive and extract piles, and are especially useful in recovering temporary steel sheeting piles. Like sonic hammers, vibratory hammers operate at quieter noise volumes, and are also useful in confined operating spaces with limited clearances.

- Powering Pile Driving Hammers

Power for the operation of a steam or pneumatic pile hammer is supplied by either a steam boiler or an air compressor. Steam boilers in general use are oil–fired. These boilers have generally replaced coal–fired boilers because of simpler fuel stocking and easier operation. Fast–steaming, lighter–weight generators – such as the McKiernan–Clayton Steam Generator – designed specifically for use in pile driving operations are available.

Portable gasoline or diesel engine–powered air compressors are commonly used for pile driving. More than one compressor may be necessary to meet the requirements of the largest hammers. Diesel pile hammers are self–contained, supplying their own motive power.

Selection of proper equipment for hammer operation is an important factor in controlling pile driving costs. Factors of importance include adequate capacity of boiler or compressor, adequate working pressure at the hammer, losses in connecting air and steam lines, and so on. The Contractor may consult the manufacturer of the hammer for specific recommendations as to power requirements of the hammer in question.

- Selecting a Hammer

  Many factors enter into the selection of the proper type and size of hammer to be used under a specific set of conditions, and this is true in the selection of all pile driving equipment. Proper equipment selection will cut the Contractor's job costs and increase job profit.

  Important factors in the selection of pile driving hammers and related equipment include the type, length, weight, and shape of the pile to be driven, the soil conditions at the job site, and the engineering Specifications pertaining to the job. Specific selection should be based upon manufacturers' recommendations and previous on–the–job experience.

  The following general principles are useful in the hammer selection process:

  - If a drop hammer is being used, experience dictates the use of a relatively heavy hammer and low fall. Such a combination will normally produce a greater penetration of the pile per blow with less damage to the pile head or cushion. The hammer should weigh at least as much as the pile it is to drive. Effective results are frequently obtained with a hammer weighing approximately twice as much as the pile.
  - When driving properly cushioned concrete piles, the application of about 25 lbs of hammer weight for every square inch of hammer weight per square foot of pile section is generally satisfactory. An engine capable of exerting a line pull in pounds of approximately twice (19.6 times) the weight in pounds of the hammer is necessary to quickly accelerate the hammer during the hoisting period.

- A single–acting pile hammer may be selected for driving conditions previously stated. As with a drop hammer, best results are usually attained if the ram of the hammer is at least as heavy as the pile being driven.
- Double–acting steam or pneumatic hammers may be used for driving any type of pile. As previously stated, their principal use is in driving sheet piles and bearing piles in common soil. Because of their high impact velocity, the pile head must be adequately protected during driving.
- Recommendations for differential–acting hammers are essentially the same as for single–acting hammers.
- Diesel hammers (most commonly used at present) offer obvious advantages of mobility and usefulness in remote or inaccessible areas, since they are not dependent on either steam generators or air compressors. McKiernan–Terry states that their Model DE–30 offers most economical driving with 1 to 3 ton piles at bearings from 40 to 90 tons, and Model DE–20, with ½ to 2 ton piles at bearings from 25 to 60 tons.
- Some manufacturers may, for a particular hammer, list both a maximum rated energy and an average working energy. For uniformity in procedure in determining that a hammer satisfies the specified requirements for size, the manufacturer's rated energy of the hammer will be used and if more than one energy rating is listed for a particular hammer, it will be the maximum energy rating.
- The energy output of a hammer may not, during the driving of a pile, be the same as the rated energy. For a double or differential acting air or steam hammer there should be sufficient air or steam pressure at the hammer to operate the hammer at the number of blows per minute required for the specified energy rating. The energy output will vary if the number of blows per minute deviates from the designated number.

Although the Contractor selects pile driving equipment that is deemed adequate to drive piles to the necessary depth and bearing without materially damaging the piles, it is advisable for the Contract Administrator to be familiar with the power plant, hammer, caps, leads, and other elements used in the pile driving operation.

The Specifications may contain the minimum energy rating of an air, steam, or diesel hammer required to drive a pile to a designated bearing. One purpose of this is to provide sufficient energy for driving the pile to the required bearing without incurring a set or penetration per blow that is so small that when it is used in the formula for determining the bearing power of the pile it may give unreliable results.

- Attachments for Pile Driver Hammers

A variety of attachments are available for use with pile hammers in order to drive different types, sizes, and shapes of piles. These include those known as base attachments, driving heads, pile caps, helmets, anvils, followers, pants, and others. Terminology is not standard throughout the industry. Use of available attachments to fit a certain hammer allows for more efficient and economical driving. In some cases, they are a necessity if the work is to be accomplished properly.

The Contractor may consult the manufacturer of the pile hammer for recommendations pertaining to attachments for use in a given set of conditions.

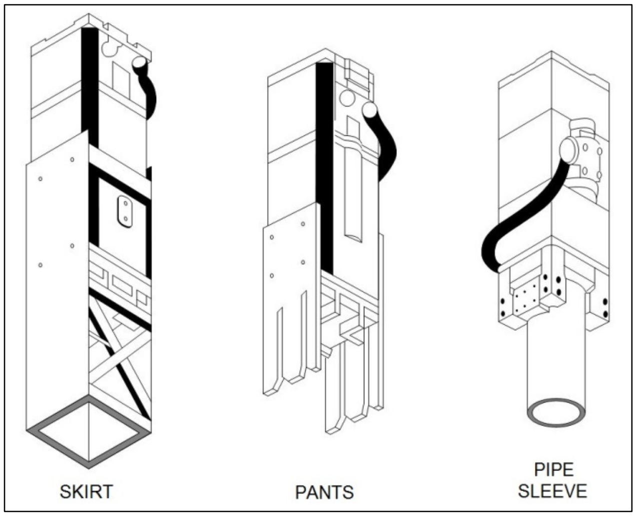


Figure 500 – 19: Special Pile Driver Attachments

- Pile Driving Leads

Pile driving leads serve as tracks along which the hammer runs, and as guides for positioning and steadying the pile during the first part of the driving operation. Leads manufactured from steel are available in a number of combinations, usually in sections from 5 to 20 ft long. Long leads may be fabricated to special order, or job–built by the Contractor.

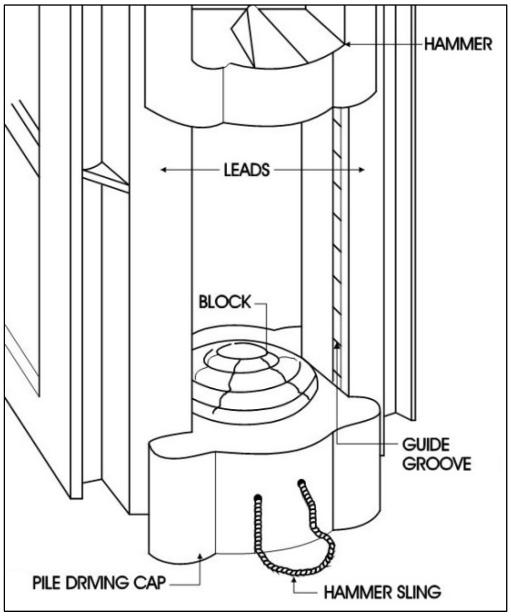


Figure 500 – 20: Assembly of Drop Hammer and Driving Cap in Bottom of Leads

Pre–fabricated leads include the following types:

Fixed Leads: Fixed leads are used to drive piles vertically or at a fixed batter. The leads are rigidly attached to the pile driver or crane.

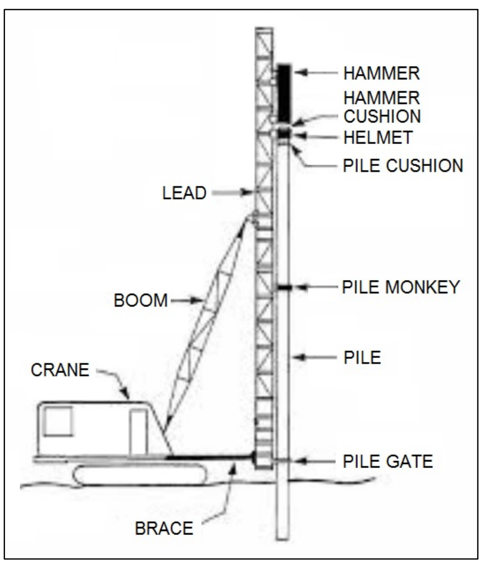


Figure 500 – 21: Fixed Lead Pile Driver Rig Configuration

Swinging (Hanging) Leads: Swinging leads are suspended from the boom point of the driver, either by a line and bail attached to the top of the leads or by boom–point adapters that are bolted to the boom–point and to the leads. Typically, the bottom of the leads is set (toed) at ground level to hold the pile's position and the leads are then plumbed or battered as required by the crane boom.

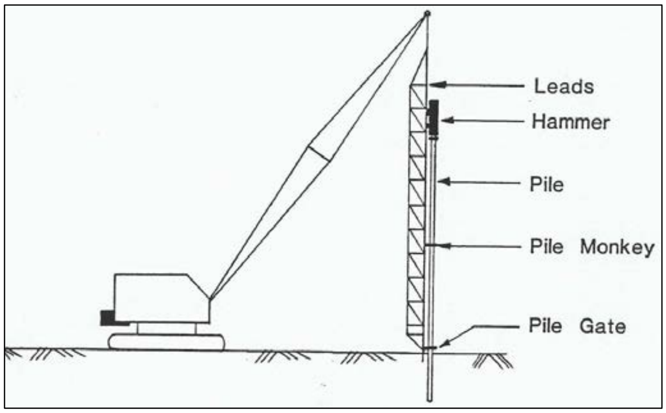


Figure 500 – 22: Swinging Leads Pile Driver Rig Configuration

Underhung vertical or batter leads hang beneath the boom–point and are held in position by lead braces that extend from the bottom of the leads to the base of the pile driver or crane. Lead braces are adjustable to attain, within limits, any desired batter.

Extended leads extend above the boom point. Leads of this type may be arranged to drive vertical or batter piles. One type, the McKiernan–Terry with extended 4–way batter leads, holds the hammer aligned with the pile at vertical and one fore, aft, right and left batter by attaching the lead to the bottom point and, through a bottom brace and moonbeam, to the cab of the crane. Leads of this type carry their own set of lead sheaves.

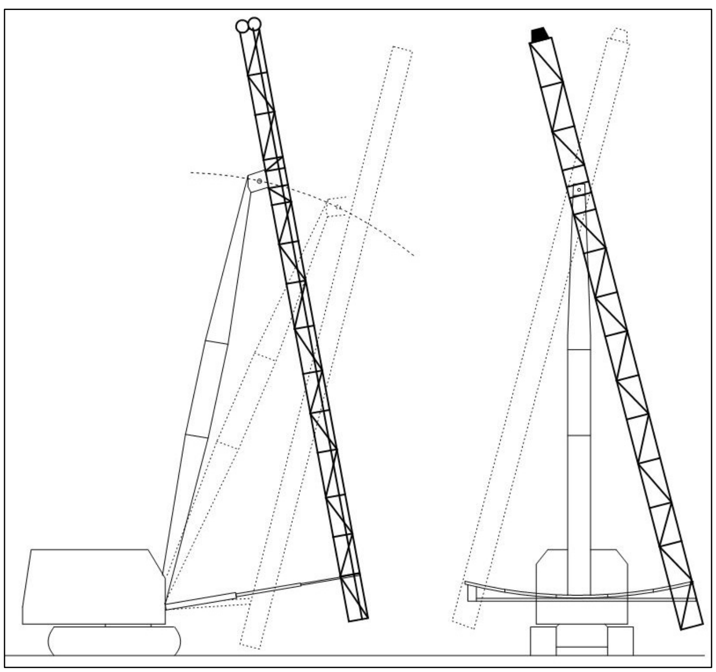


Figure 500 – 23: McKiernan–Terry 4–Way Batter Leads

- Attachments for Leads

Extension Above Boom Point: An extension above boom point is a design variation that can be applied to 6:1 and 3:1 fore and aft batter leads. It permits the use of a shorter crane boom with a correspondingly greater crane capacity than is obtainable with a full length boom for the specified lead length. It consists of a special boom hook section and necessary extension sections. A sheave head is built into the uppermost section.

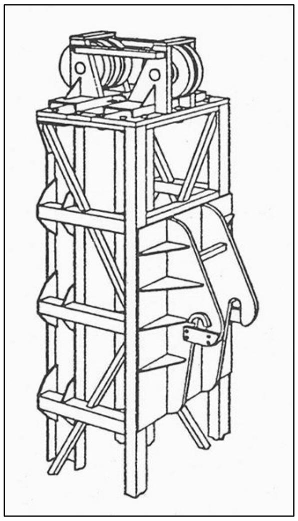


Figure 500 – 24: Extension above Boom Point Attachment

Hairpin: A hairpin provides an easy means of entering and removing a hammer from the leads. It can be blocked at any point in the leads permitting the hammer to slip below it and out of the leads without removing the angle–iron guides or dropping the hammer in a hole. A hairpin also provides a means of using a narrow hammer in leads that are also used for a wide hammer.

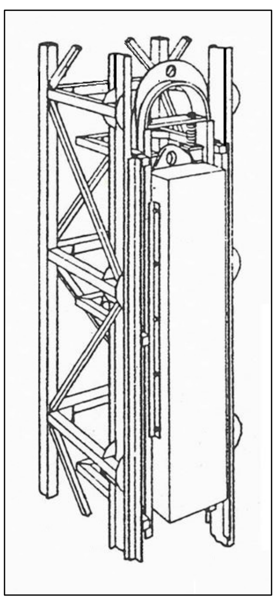


Figure 500 – 25: Hairpin Lead Attachment

Telescope: A telescope is used to provide an extension below the lead to permit supported operation of the hammer below grade.

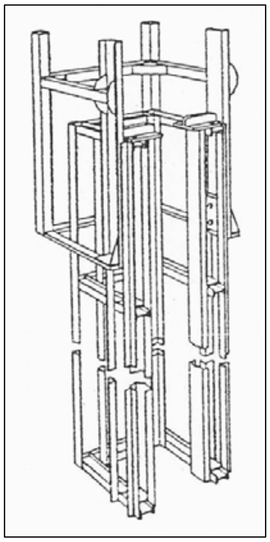


Figure 500 – 26: Telescope Lead Attachment

- Jetting Equipment

As an alternative to driving piles, “pile jetting” is a technique for placing piles that uses pressurized water applied as a concentrated jet at the pile tip. The soil immediately below the jet is liquefied and the bearing capacity of the soil is greatly reduced. Friction is also eliminated, and the pile slides into the hole excavated by the jet, usually by its own weight.

Jetting equipment consists of standard steel pipe and pipe fittings that are made into a jetting assembly. Flexible water hose and couplings connect the jetting assembly to a pump, and the jetting assembly may be handled by the same rig used to handle the piles.

Jetting pipes usually have an inside diameter of 2½ to 3½ in. For use in gravelly soils, water pressure should range from 100 to 150 psi. For sand, water pressure from 40 to 60 psi is generally adequate. Piles may be jetted by attaching pipes to the sides of the pile.

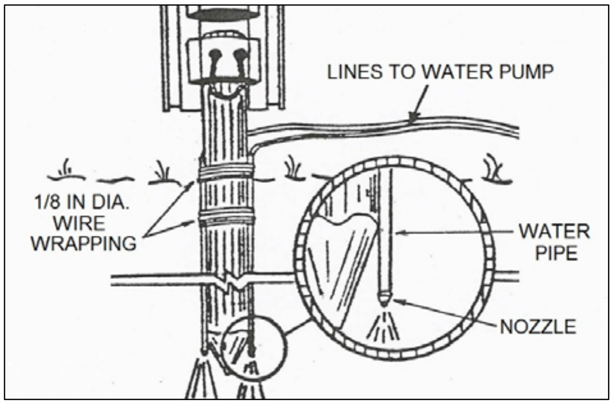


Figure 500 – 27: Suggested Method for Jetting Piles

- Pile Extractors

  Pile extractors are available from manufacturers of pile hammers. They are essentially specially designed double–acting hammers operated by steam or air. Extractors are furnished in various sizes suited to the type of pile to be pulled and the lifting capacity of the crane. It is always desirable to exert as much upward pull on the pile as possible. Therefore, an extractor should be chosen that is capable of withstanding the maximum pull that can be exerted by the crane. Various attachments are available for use with the extractor in pulling different types of piles.

  If desired, certain double–acting hammers can be rigged in an inverted position with a wire rope sling and used as pile extractors. Generally speaking, the use of this method is not as efficient as is the use of an extractor. The following table lists the specifications of two types of McKiernan–Terry extractors.

Specifications of Two Types of McKiernan–Terry Pile Extractors		
Extractor Size Number	E2	E4
Net Weight of Extractor and Attachment (lbs)	2600	4400
Weight of Ram (lbs)	200	400
Bore (in)	7	9
Stroke (in)	3	3
Energy of blow (ft–lbs)	700	1000
Strokes per minute	450	400
Width, overall (in)	25	26
Depth, overall (in)	19	22
Length, overall (in)	100	125
Diameter of pile clamp bolt (in)	2 ⅜	2 ⅜
Width of standard pile clamp (in)	6	6
Air consumption, actual (ft3/sec)	400	550
Boiler power (hp)	30	35
Hose connection (in)	1 ½	1 ½
Maximum crane pull (tons)	50	100

Note: Steam or air pressure should not exceed 125 psi gauge pressure.

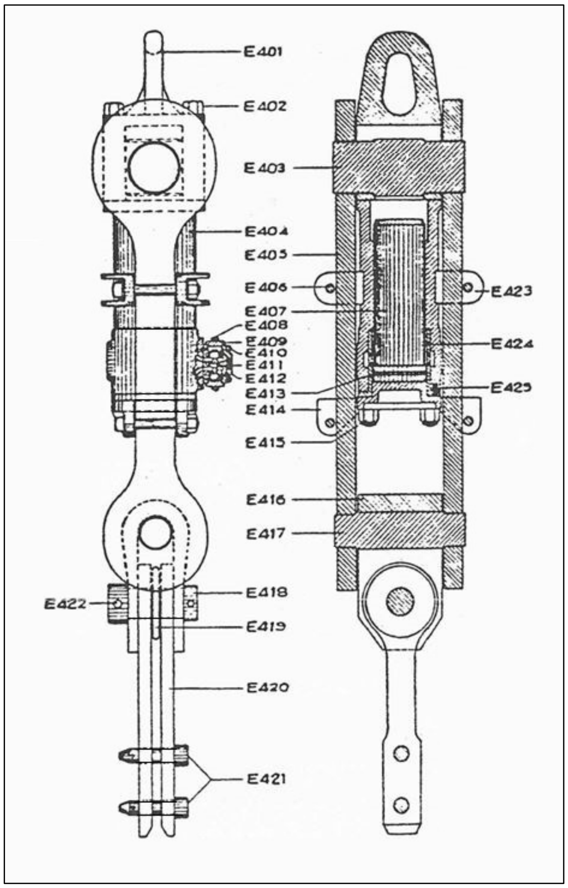
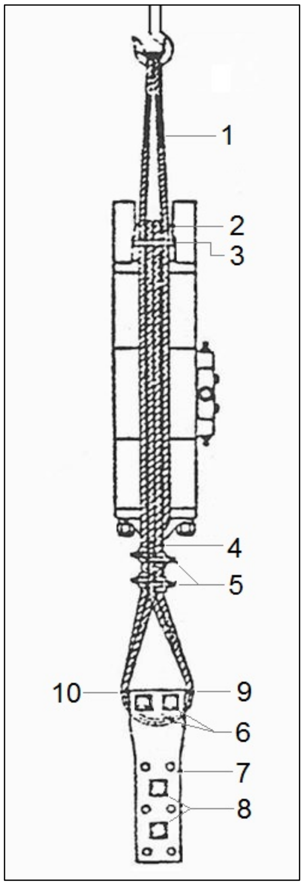


Figure 500 – 28: McKiernan–Terry Extractor Attachment Schematic Diagram

The following diagram illustrates a McKiernan–Terry cable pulling rig for using a pile hammer to extract piling. The rig shown was devised for use with McKiernan–Terry Nos. 5, 6, and 7 hammers.



Cable–Pulling Rig Parts		
Part No.	Name	No. Req.
1	Crane Sling	1
2	Grooved Saddle Block	1
3	Restraining Ring	1
4	Pile Sling	1
5	Clips	8
6	Pile Sling Bolt & Nut	2
7	Pile Clamp	2
8	Pile Bolt & Nut	2
9	Half Sheave (Single)	2
10	Half Sheave (Double)	1

Figure 500 – 29: McKiernan–Terry Cable Pulling Rig for Pile Extraction with Parts List

S. Wave Equation Analysis

Wave equation analysis is a numerical method of analysis for the behavior of driven foundation piles which predicts the pile capacity versus blow count relationship (bearing graph) and pile driving stress. The model mathematically represents the pile driving hammer and all its accessories (ram, cap, and cap block), as well as the pile, as a series of lumped masses and springs in a one–dimensional analysis. The soil response for each pile segment is modeled as viscoelastic–plastic. The method was first developed in the 1950s by E.A. Smith of the Raymond Pile Driving Company.

Traditional dynamic formulas, such as the Engineering News Record Formula, do not provide accurate predictions of actual pile capacities and they provide no information on stresses in the piles. The one–dimensional wave equation analysis has eliminated many shortcomings associated with the pile driving process.

The wave equation analysis uses wave propagation theory to monitor the longitudinal wave transmitted along the pile axis when it is struck by a hammer's impact. As the ram impact occurs, a force pulse is developed in the pile that travels downward toward the pile tip at a constant velocity that is dependent on the pile's material properties. When the force pulse reaches the portion of the pile that is embedded, it is attenuated by soil frictional resistance along the pile. If the attenuation is incomplete, the force pulse will reach the pile tip and a reflected force pulse, which is governed by the soil tip resistance, is generated. The pile will penetrate into the soil when the peak force generated by the ram impact exceeds the ultimate soil resistance at the pile tip.

The wave equation analysis provides two types of information:

- The analysis provides a relationship between pile capacity and driving resistance. The user inputs data on soil side shear and end bearing and the analysis provides an estimate of the set [in/blow] under one blow of the hammer. By specifying a range of ultimate pile capacities, the user obtains a relationship between ultimate pile capacity and penetration resistance [blows per 1 in or blows per 10 in].
- The analysis also provides relationships between driving stresses in the pile and penetration resistance.

The wave equation analysis enables the user to develop curves of capacity versus blow count for different pile lengths and to use these in the field to determine when the pile has been driven sufficiently to develop the required capacity. The wave equation also provides results for blow count versus stroke for a particular pile capacity.

The analysis is used to select the right combination of driving equipment to:

- Ensure that the piles can be driven to the required depth and capacity
- Design the minimum required pile section for driving
- Minimize the chances of over–stressing the pile
- Minimize driving costs

A static analysis is performed in order to provide input data for the wave equation analysis. The input data generated by the static analysis includes pile length, load transfer distribution, and the ultimate load capacity.

The Contractor’s pile driving equipment summary should be provided to the Bureau of Materials and Research Soils Engineer at least 30 days prior to pile driving so that a wave equation analysis can be conducted. The Soils Engineer should also visit the site during the initial stages of pile driving to verify the applicability of the wave equation analysis driving criteria.

T. Pile Dynamic Load Testing

Technical services and equipment for dynamically testing piles are available through the Geotechnical Section of the Materials and Research Bureau. This work should be scheduled and coordinated through the Soils Engineer.

The testing equipment for dynamic testing includes these components:

- A pair of strain transducers mounted near the top of the pile
- A pair of accelerometers mounted near the top of the pile
- A pile driving analyzer (PDA)

The pile driving analyzer monitors the output from the strain transducers and accelerometers as the pile is being driven and evaluates the data as follows:

- The strain data, combined with the modulus of elasticity and cross–sectional area of the pile, gives the axial force in the pile.
- The acceleration data, integrated with time, produces the particle velocity of the waves traveling through the pile.
- The acceleration data, double integrated with time, produces the pile set per blow.

Figure 500 – 30: Pile Dynamic Load Testing

These dynamic measurements are used to evaluate the performance of the pile hammer; calculate pile installation stresses; determine pile integrity; and estimate static pile capacity. Dynamic test results can be further evaluated using signal matching techniques to determine the relative soil resistance distribution on the pile as well as representative dynamic soil properties for use in wave equation analysis.

In general, dynamic load testing should be conducted on the first piles driven for a substructure to evaluate the driving system efficiency, to evaluate the dynamic pile capacity and to check pile stresses. The dynamic load test should also be conducted on selected piles as necessary, particularly if there is a question of pile damage.

- Pile Static Load Testing:

Static load tests may be required on certain projects and are usually specified in the contract. Static load tests should be conducted early in the construction phase and, if possible, before pile lengths are ordered. A static load test may also be required if the driving conditions encountered during construction are not consistent with the anticipated subsurface conditions and if dynamic measurements cannot give a reasonable interpretation of the pile capacity.

The main purpose of the load test is to verify that the actual pile response to an applied load corresponds to the response assumed by the designer and that the actual ultimate pile load is not less than the computed ultimate load used for design. The static load test can result in foundation cost savings, particularly on a friction pile project.

The setup, monitoring, and interpretation of load tests should be coordinated through the Soils Engineer. Refer to *ASTM D–1143 Standard Test Methods for Deep Foundations Under Static Axial Compressive Load* for information and testing methods to follow for compression load testing of piles.

This procedure involves applying a compression load to the pile and measuring movements. Most often, compressive loads are applied by hydraulica1ly jacking against a beam that is anchored by piles or ground anchors, or by jacking against a weighted platform. The load should be measured using a calibrated pressure gage and calibrated load cell. Axial pile head movements are usually measured by dial gages that measure the pile head movement relative to an independently supported reference beam.

The following illustration shows various jacking load test conditions.

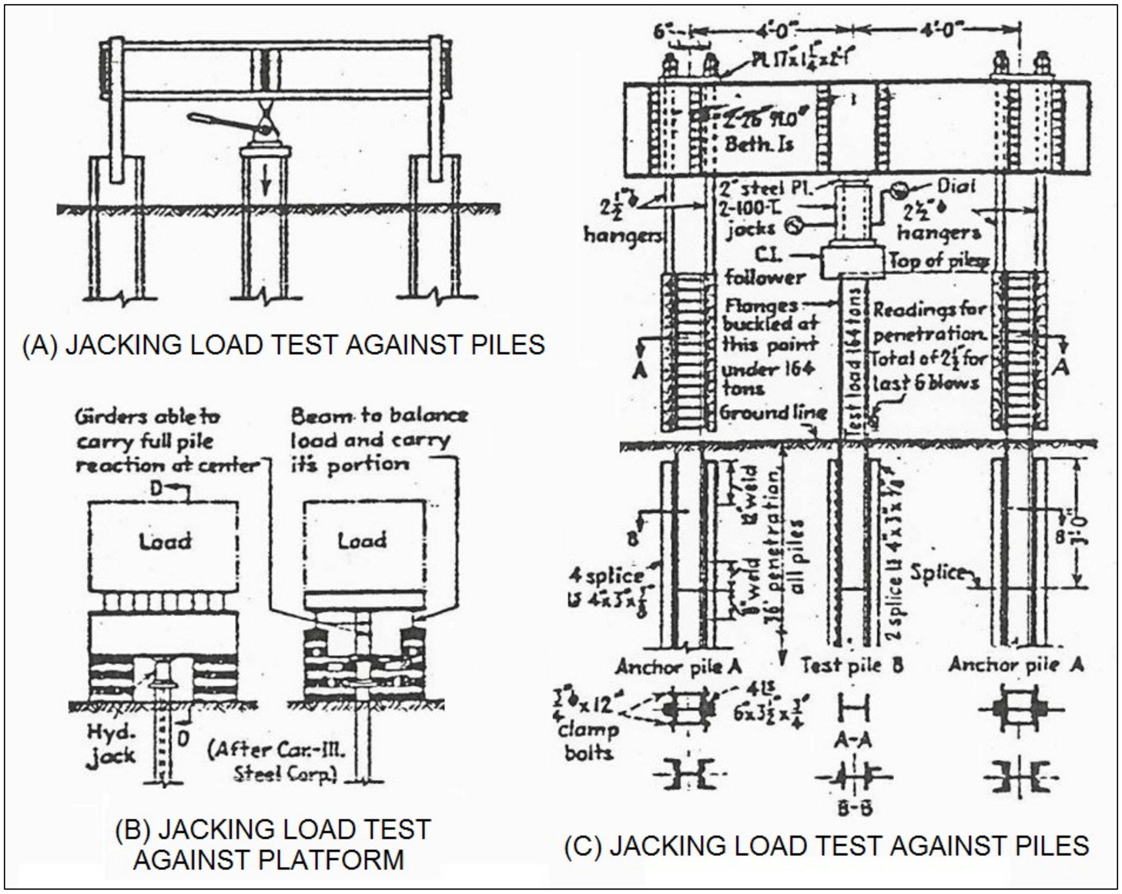


Figure 500 – 31: Jacking Load Test Conditions for Piles

The following illustration shows various cantilever load test conditions.

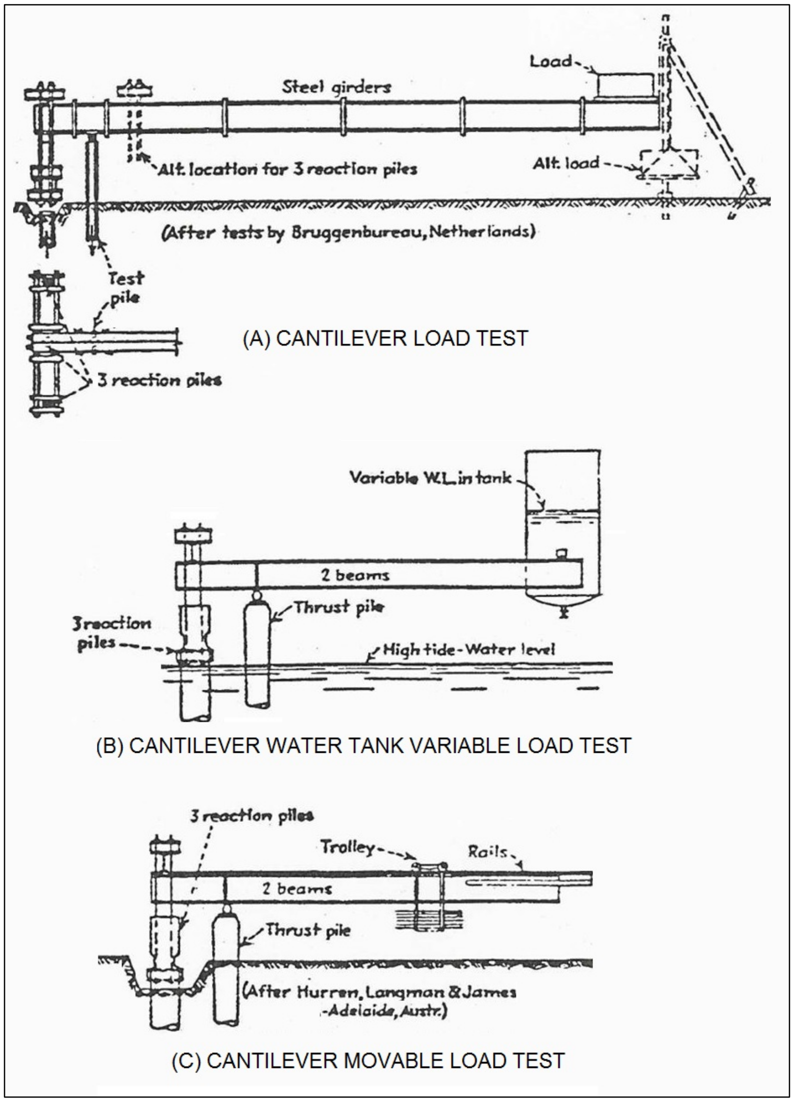


Figure 500 – 32: Cantilever Load Test Conditions for Piles

The following illustration shows various platform load test conditions.

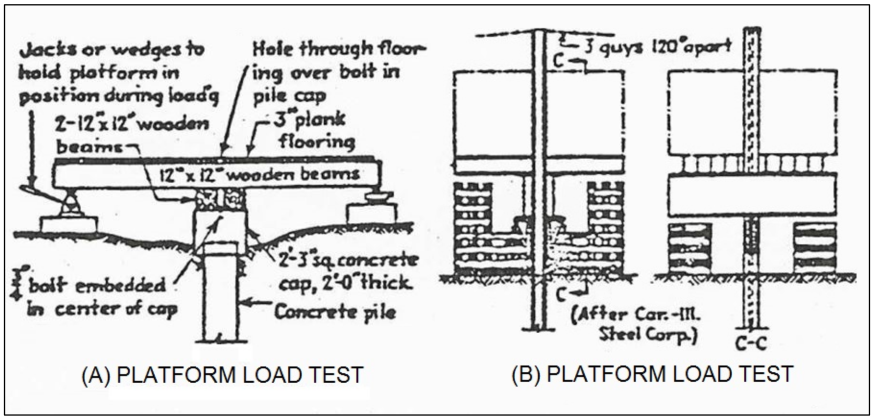


Figure 500 – 33: Platform Load Test Conditions for Piles

U. Characteristics of H–Piles in Various Soils and when Driven to Rock

The subject of soil mechanics and the relationship of this subject to foundation piles have been thoroughly discussed in a number of books and articles. This subsection reviews some of the broader principles on the specific subject of H–piles.

Soils are classified according to size by the U.S. Department of Agriculture, as follows:

USDA Soil Classifications	
Soil Type	Diameter (in)
Gravel	Greater than 0.08
Sand	0.002 – 0.008
Silt	0.001 – 0.002
Clay	Less than 0.0001

In natural beds, soil occurs, as often as not, as a mixture of two or more of the above classifications. Thus, we commonly see soil described as sandy clay, silty sand, clayey sand, etc. The problem is further complicated for the engineer designing a pile foundation by the fact that soils usually occur in layers of differing classifications, frequently with many layers at the site of a particular project.

A foundation pile driven entirely into soil derives its load–carrying capacity from two sources, namely, point resistance and skin friction.

Point resistance is defined as the resistance of the soil to being pushed aside to make way for the pile. This action occurs at and around the end of the pile in the case of a pile with parallel sides. A similar action occurs elsewhere along the pile if the pile is tapered or it's an H–pile with special attachments.

Skin friction, as its name implies, is the resistance of the soil to sliding along the surface of the pile.

For the purpose of this section, soils and subsoil conditions shall be divided into four groups, and each discussed in relation to H–piles. These classifications are hard rock; shale, hardpan, marl, and soft rock; sand, gravel, sand and gravel; and clay.

- Piles Driven to Hard Rock

  Steel is the only material used for foundation piles that has an ultimate strength in compression comparable to that of hard rock. This fact results in an entire field of foundation structures for which H–piles are predominantly suitable. For example, in the case of bridge piers founded on rock lying at great depth below water level, H–piles have been used in lengths close to 200 ft, carrying extremely high loads per pile. These piles, driven on close centers, are capable of taking advantage of the high bearing capacity of the rock in a manner not possible with other types of piles.

In a majority of cases, the surface of hard rock is covered by a thin layer, 4 inches or more in thickness, of soft disintegrated rock. The H–piles penetrate the soft layer, and become firmly seated on the hard rock. If the rock is overlain by hardpan, sand and gravel, stiff clay, or other hard material, the condition is very satisfactory for fixing the point of the pile for bearing on the rock.

In all such cases, it is not necessary to incur the expense of reinforcing the point of the pile. In the occasional case of extremely hard ledge rock immediately overlain by very soft material such as mud, consideration may be given to reinforcing the point of the pile.

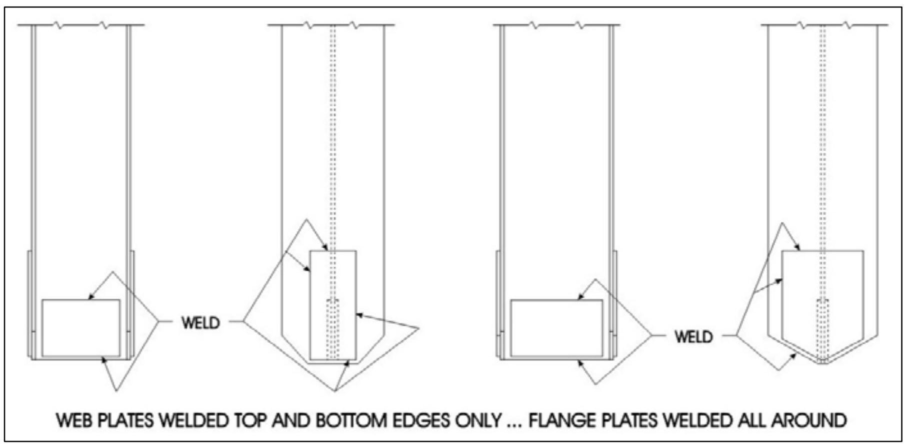


Figure 500 – 34: Types of Pile Point Reinforcement for H–Piles

- H–Piles in Shale, Hardpan, Marl, and Various Soft Rocks

No discussion of the mechanics of this class of material is necessary, since it is readily admissible that point resistance, or in some cases, point resistance plus skin friction over a short length of the pile will develop full bearing capacity. H–piles are particularly suitable in this situation because high load concentrations are possible. The smaller number of H–piles, each carrying a greater load than is feasible with most other types of piles, will produce economies not only in the piles themselves but in other directions.

The fact that H–piles can withstand sustained hard driving makes it possible to obtain deeper penetration in this class of material than is possible with piles of bulky cross–section. This is important, since it is possible to firmly fix and seat the piles.

- H–Piles in Sand, Gravel, or Sand and Gravel

The chief properties of sand, gravel, and sand and gravel, aside from the size of individual particles, are permeability, incompressibility (unless in a loose, uncompacted state), high coefficient of friction, and low cohesive strength. These properties are directly opposite to those of clay, which are dealt with in a subsequent section.

Since soils comprised of compact sand, gravel, or sand and gravel are incompressible to a high degree, the principal action at the point of the pile is to push aside the soil to make room for the pile. There is little compaction of the soil, and water is not squeezed out of the voids as in the case of clay. When driving ceases, the soil is already in a state of equilibrium, and the point resistance of the pile remains constant.

In compact sand, skin friction is not decreased by water lubrication during driving, as in the case of clay, since there is no tendency for the water to escape. When driving is discontinued, the pressure of the soil against the pile is about the same as during driving, and the skin friction resulting there from becomes an important source of load–carrying capacity.

H–piles are especially suitable for use in compact sand, gravel, or sand and gravel. Because these soils are highly incompressible, the displacement resulting from driving an H–pile, though such displacement is comparatively small, is ample to develop a high intensity of compressive stress both at the end point and along the sides of the pile, with correspondingly high values for point resistance and skin friction.

The resistance of sand and of gravel to being pushed aside by a pile is so great that it usually is impossible to obtain more than a small amount of penetration with the type of pile that has a solid cross–section and large bulk. In many instances, a certain minimum amount of penetration is just as important as load–carrying capacity. This is often so in the case of bridges where the river bottom is subject to deep scour and it is imperative to drive the piles to sufficient depth to protect against such action.

H–piles are in a class by themselves in such cases. They can be driven further than other types of piles because of their small displacement. Their ability to take extreme punishment allows them to stand being driven long after they have reached a depth where ample bearing capacity has been developed.

Adequate bearing capacity normally is reached in this class of soils with a moderate amount of penetration by the use of plain H–piles, and it is, therefore, unnecessary to modify the piles with any attachment. In occasional cases of very fine saturated sand, or where the sand is mixed with a minor percentage of clay, or of loose sand that is subject to compaction due to rearrangement of the particles under vibrating loads, the penetration of plain H–piles may be deemed excessive.

It may become economically worthwhile at this point to increase the bulk of the piles with suitable attachments, commonly called lagging. This should be necessary for only a comparatively short distance 8 to 12 ft and the lagging should be located near the top of the load–bearing stratum. If located near the bottom, it will have the effect of destroying the skin friction of the piles above the lagging. This is due to the transition from the bulky cross–section to the plain H–pile relieving the soil of the compressive stresses that produce skin friction.

- H–Piles in Clay

The chief characteristics distinguishing clay from other soils are the smallness of the particles; a high degree of impermeability and compressibility; a low coefficient of friction; and cohesive strength that may vary from 87 psi or less for liquid clay, to 14 psi for very stiff clay. These characteristics vary, generally speaking, with the moisture content. The engineering properties deteriorate as the percentage of moisture increases. This does not mean that the moisture content is a definite index to the properties of all clays, as two different clays with the same moisture content may differ decidedly as to properties.

Clay is compressible, and this is true to a far greater degree than in the case of sand or gravel. When pressure is applied, water is squeezed out of the voids, and the solid particles are pressed into closer contact. The water naturally escapes through pathways of least resistance. Skin friction during driving of an H–pile in clay may be small. The soil, which has been forced aside, exerts pressure against the sides of the pile, but since the pile is in motion, the small coefficient of friction of the clay results in small frictional resistance. The coefficient of friction will even be reduced below its normal value by lubrication caused by water squeezed out of the clay by compression at the pile point and lateral compression due to vibration of the pile.

After driving is completed, however, the lateral pressure, which has been set up during driving, forces the soil particles into close contact with the comparatively rough surface of the pile, resulting in a strong bond. It is this bond that furnishes the chief means of transmitting a load from the pile to the soil. In many cases, the bond is stronger than the shearing resistance of the soil. This is demonstrated when an H–pile in clay is pulled. It is not unusual for the pile to come up with the spaces between flanges and web filled with cores of soil.

The great difference that may exist in clay between driving resistance and static resistance under load is well known. All those acquainted with pile driving are familiar with the phenomenon of a pile that drives easily in clay but after a period of rest sets up solidly. Upon redriving such a pile, the initial penetration per blow is considerably less than the final penetration for the original driving. In the case of plain H–piles, the static resistance almost always will be greater than what the driving resistance indicates because the skin friction increases from what may be a negligible value during driving to a substantial value after rest.

In hard, stiff clay containing only a small percentage of moisture, the compressibility will be small, and therefore the amount of displacement and compression required to develop its full capacity will be correspondingly small. When an H–pile is driven into such clay, it frequently happens that the soil trapped between the flanges and web becomes so hard due to compression that it grips the pile and is carried down with it. The pile thereupon becomes in effect a displacement pile, and the core of soil trapped on each side of the web performs the same function that lagging serves in softer soil. Under such conditions, plain H–piles will develop very satisfactory load–carrying values.

In the more plastic compressible clays, the bearing value of a plain H–pile may be small, and consideration must be given to the lateral compressibility of the soil in relation to the cross section and bulk of the pile. In such soils the proper use of H–piles may necessitate the attachment of suitable lagging to the piles to increase their displacement and thereby develop more fully the bearing capacity of the soil. This type of pile has an advantage over a conventional displacement pile where extremely long lengths are required, where there is lateral impact (as in water–front structures), or where there is a long unsupported length.

The lagging should be attached throughout the depth of the load–bearing stratum, instead of for a yard (meter) or so as in the case of loose saturated sand. If the clay is overlain by soft soil of no bearing value, nothing is gained by lengthening the lagging to include such soft material. The lagging provides the large section area at the point of the pile which is necessary to obtain the degree of displacement and compression to develop the full resistance of the soil, and it also maintains the pressure along the sides of the pile necessary for skin friction.

V. Lagging

Lagging is attached to an H–pile to increase its cross–section and thereby its displacement of the soil. The purpose of this is to increase the bearing value of the pile. Its use is considered only where the bearing value must be developed in plastic, cohesive soil, and occasionally in loose, uncompacted, fine sand. It is of no value where the pile is driven into compact sand, sand and gravel, hardpan, or soft rock.

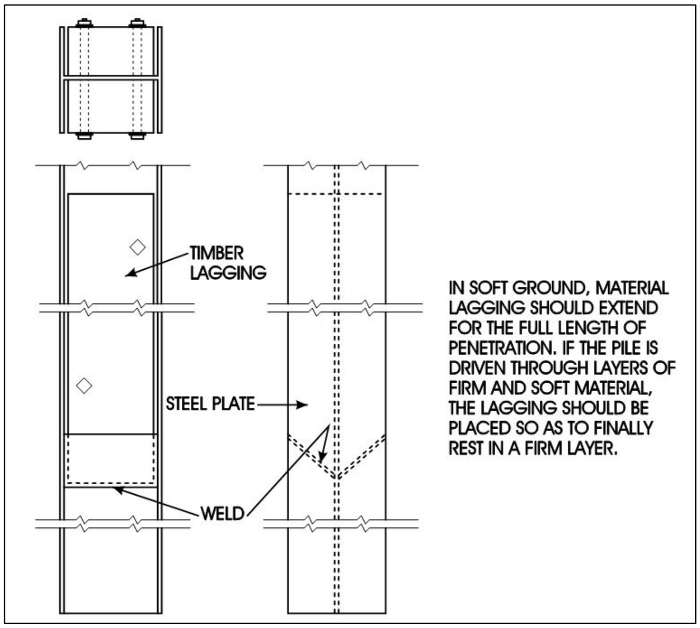


Figure 500 – 35: Lagging for H–Piles

If possible, comparative load tests of lagged and unlagged piles should be made to determine definitely whether the lagging is economical. The fact that an unlagged H–pile drives easily through plastic cohesive soil does not necessarily mean that it has no bearing value under such conditions. After driving ceases, the pile may set up a bond with the soil that will develop a satisfactory load–carrying capacity. For additional discussion of lagging, see the previous subsection covering H–piles in clay.

W. Pile Point Reinforcement

Pile point reinforcement should be used in cases where the piles are driven to hard rock. Pile point reinforcement never should be used where the piles must develop skin friction, since the enlarged point will greatly reduce the skin friction on the surface of the pile above the point. Reinforcement is specified by some engineers to distribute the load over the rock in order to reduce the unit stress, although there is some question regarding the necessity for its use for this purpose, as discussed previously under the subsection covering H–piles driven to rock.

Another reason to use pile point reinforcement is to protect the point of the pile from buckling when driven to hard rock. To avoid buckling, the piles should not be overdriven. Although commercially available, pile points are routinely ordered and welded onto the leading ends of the piles.

Note: The use of pile tips is decided in the design stage and is not generally a decision made in the field.

X. Pile Driving Formula

Generally speaking, where the size and nature of the project will permit, loaded test piles should be used instead of formulas for determining the safe load per pile. Engineers tend to keep the load per pile low where a formula must be depended upon due to the uncertainties inherent in the use of a formula. Substantial economies can often be affected by increasing the load per pile based upon the reliable information obtainable from properly conducted pile load tests.

Pile–driving formulas are particularly useful for small projects where the cost of load tests would be an important percentage of the total cost of the piles required for the project.

Example

The theoretical elastic shortening of the pile may be calculated based on the pile's length, cross–sectional area, and Modulus of Elasticity, along with the test load.

Given:

12x53 Steel H–Pile:

12 in depth

53 lbs/ft

65 ft long

30 ton pile

60 tons test load

e = Theoretical Elastic Shortening of Pile (in)

P = Test Load = 60 tons

L = Length of Pile (in) = 780 in

A = Cross–sectional Area of Pile (in^2) = 15.6 in^2

E = Modulus of Elasticity = 2.9×10^7 lbs/in^2

Solve:

$$e = \frac{\text{Load} \times \text{Length}}{\text{Area} \times \text{Modulus of Elasticity}}$$

$$e = \frac{PL}{AE}$$

$$e = \frac{[(60\,tons)(2000\,lbs/ton)] \times [(65\,ft)(12\,in/\ ft)]}{(15.6\,in^2) \times (2.9 \times 10^7\,lbs/in^2)}$$

$$e = \frac{(120{,}000\,lbs) \times (780\,in)}{4.524 \times 10^8\,lbs}$$

$$e = 0.2069\text{ in}$$

Therefore, the theoretical elastic shortening of the 12x53 H–Pile under a 60 ton load is 0.2069 in.

SECTION 520 – PORTLAND CEMENT CONCRETE

520.1 – GENERAL

The purpose of this section in the manual is to provide information to supplement the Specifications with respect to the item of concrete. The five classes of concrete are designed primarily on the basis of a presumed strength. No matter what class of concrete is involved, the variety of inspection duties that the Contract Administrator must perform is generally the same. The item of concrete includes forming, placement, finishing, and curing. All aspects must be accomplished properly for successful concrete masonry.

Figure 500 – 36: Placing Concrete in Approach Slab

520.2 – MATERIALS

All of the concrete used on NHDOT projects comes in transit mixers from approved concrete batching plants. The Specifications are comprehensive in detailing requirements of the batch plant and mixers that must be met before they are approved by the Materials and Research Bureau. Therefore, control and inspection of the plant and materials at the plant consists of only the testing and sampling that is done by the Contract Administrator's representative, the Concrete Plant Inspector.

The Concrete Plant Inspector's testing and sampling procedures are detailed in the following subsections. As previously stated, personnel from the Bureau of Materials and Research perform periodic checks of the plants and truck mixers supplying concrete for Department projects. However, the Concrete Plant Inspector should be knowledgeable of the Concrete Plant Specifications for overall understanding of concrete batching.

520.3 – CONSTRUCTION OPERATIONS

A. Review of Plans

The Contract Administrator shall review the plans and make the necessary computations to verify concrete quantities for the record book. Most concrete items are final pay (F) items, so exhaustive computations of the concrete quantities are not necessary. However, a rough check should be made to assure that there is no gross error (±25 %) made in the calculation of design quantities. The verification of quantities should be completed before payment. Field personnel should become familiar with all aspects of the planned concrete placement operations, including the configuration of the structures and the concrete pour volumes.

Ensure that the proper class of concrete is specified for the different sections of the structure. For example, all abutment backwalls shall be Class AA concrete or the same mix design as the deck, as stated on the plans. If this is not indicated on the plans, the necessary corrections should be made.

Well in advance of grading the bridge seat forms, the Contract Administrator should calculate the depth of materials and verify the bearing area elevations with the finished roadway grade. A negative blocking distance on parts of the span may indicate an increase in bearing area elevations, a delivered beam with excessive camber, or the planned dead load deflection not being totally realized. The Contract Administrator should discuss any problems of this type with the District Construction Engineer and have the bridge seats lowered to eliminate the occurrence of any resulting humps in the deck, curb, sidewalk, or guardrail.

B. Materials and Research Bureau

As soon as the Contractor has designated the concrete supplier, design mixes should be submitted by the supplier to the concrete lab at the Materials and Research Bureau in Concord. The Contractor should notify the Contract Administrator of the supplier selection, and the Contract Administrator should be available to the supplier and the concrete lab to handle any issues regarding the design mix submittals. The design mixes for all classes of concrete that are planned for use on the project must be approved by the Materials and Research Bureau prior to any concrete placement activities.

Sample Concrete Mix Design forms may be found in Section 705 Sample Forms from the Bureau of Materials and Research in this Manual.

C. Design Mix

The design mix received from the lab will typically provide concrete acceptable for all uses on the project based upon historical data. However, there are times when workability or the advantages of admixtures may require field adjustment of the mix for specific pours.

Should the concrete mix design be repeatedly unsatisfactory, the Contract Administrator should have the supplier redesign and resubmit the mix after a complete evaluation rather than continue with field adjustments. Adjusting the water and aggregate in the mix will only be done after testing in the field proves the designed mix is not meeting requirements. The factors determining these adjustments will be discussed in the field testing section.

The proper use and dosage of admixtures such as air–entraining, water–reducing, and retarding admixtures is generally determined from previous experiences. The initial starting dosages are recommended by the supplier for the intended use and may need adjustment once the concrete is tested. If there is any question as to their uses and the quantities to add, the District Construction Engineer or other field personnel with experience in this area should be consulted.

Ordinarily, using the air meter field test will disclose whether or not enough air entraining admixture is being incorporated into the mix. Adjusting the amount up or down by fractions of an ounce per cubic yard can usually be called in to the plant over the transit mix truck's radio or by telephone until further testing of adjusted mixes shows the proper air content.

Redosing the air entraining agent in the field will enable a load with low air content to be used, but this responsibility should always be restricted to qualified quality control personnel from the supplier. Neither the Contract Administrator nor NHDOT personnel should make dosage recommendations, only check for compliance.

Regarding the use of retarding admixtures, factors to be considered include the weather, the time it will take to place the quantity of concrete, and the Contractor's concrete placing performance, including their ability to finish the concrete once placed. The District Construction Engineer and/or other field personnel should be consulted regarding the amount of retarding admixture to include in the mix to start a pour and the stages at which this amount should be reduced.

The following figure is a graph depicting the relationship between temperature and time to set the concrete when using retarding admixtures.

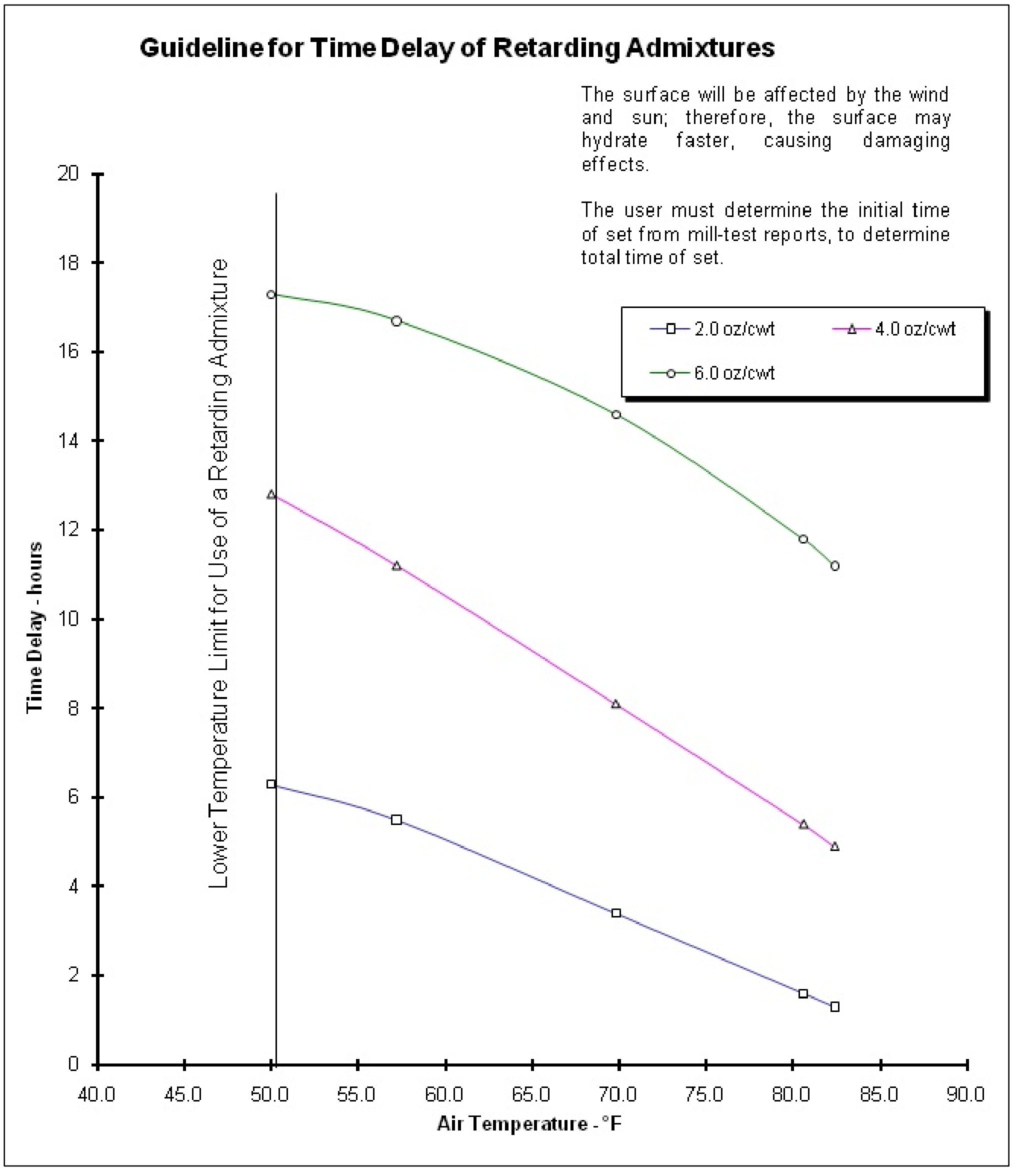


Figure 500 – 37: Effects of Retarding Admixtures on Concrete Setting Time

Approved concrete admixtures may be found on the Materials and Research Bureau's Qualified Products List.

D. Subgrade Preparation

Subgrades should be graded to their specified elevations and should be moist when the concrete is placed. A moist subgrade is especially important to prevent rapid extraction of water from the concrete when footings or slabs are being placed. Where the foundation is rock, all loose material should be removed before the concrete is placed. Pressure washing or compressed air are recommended methods of cleaning. In general, when it is necessary to cut out ledge or rock the surfaces should be vertical or horizontal (or in combination), not sloping.

E. Checking Concrete Formwork

The Specifications are detailed in discussing formwork. Forms should be clean, tight, adequately braced, and constructed of materials that will impart the desired texture to the finished concrete. Plywood panels should be assembled with the grain running horizontally. Care should be taken to see that sawdust, nails, and other types of debris are removed from the sections to be placed.

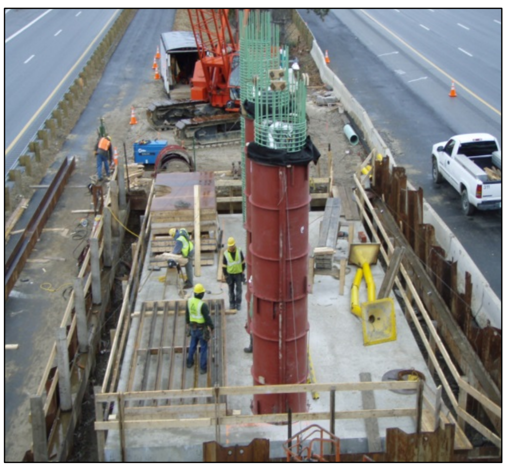


Figure 500 – 38: Concrete Formwork for a Column

Usually, during the excavation operations or preparation of the subgrade, control points or offsets are constructed to locate the centerlines of bearing and roadway. The Contract Administrator must be confident that these offsets are correctly located and use these points to check the assembly and verify the location of the forms. The forms must be set vertically, horizontally, or battered as specified and securely braced to stay true. Trueness should be continually monitored throughout the placement with the use of plumb lines or other accepted methods.

After the formwork is completed by the carpenters, the Contract Administrator should verify that the formwork is properly sized and configured, and will be strong enough to resist deformation under the weight of the poured concrete. In addition to line, the Contract Administrator will verify the elevation of the top of the pour at several different locations on the form.

The bench marks should be the same, relative to each other, and should be checked against each other periodically. In addition to checking this elevation against several bench marks on the job, it is advisable to set up a level immediately after the pour and check the top of the concrete elevation again. This checking procedure must be performed with precision when pouring bearing areas with precise finished elevations.

Finally, when the forms are constructed and the reinforcing steel is tied, the clearance between the steel and the outside of the form and top of the concrete must be checked. It is important to have the proper clearances as shown on the plans. Using templates or string lines to illustrate the finished concrete grade will provide a method to check clearance of top mats in sidewalks and walls.

F. Stone Masonry Formwork and Chamfer Strips

Two items of particular interest with regard to formwork are emphasized in this subsection: Stone masonry formwork and chamfer strips.

Figure 500 – 39: Concrete Stone Masonry Installation

- Stone Masonry Formwork

There is a special consideration when constructing forms intended to hold concrete against stone masonry walls. Usually, the face of the wall has been built using irregular quarry or field stone behind which the concrete will be placed. Naturally, support to each individual stone must be provided by some sort of formwork so that the weight of the plastic concrete being placed against them will not dislodge the stones and cause a complete collapse.

An ideal approach to forming and supporting a concrete wall incorporating a stone masonry face is shown in the following figure.

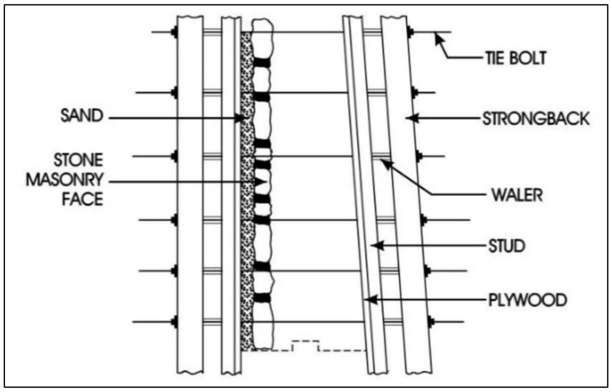


Figure 500 – 40: Sand Method of Securing Stone Masonry Faces for Concrete Placement

However, few Contractors consider this application economically feasible and, therefore, use some combination of walers and wedges in lieu of a sand cushion. Each stone usually requires at least two wedges, as shown in the following figure.

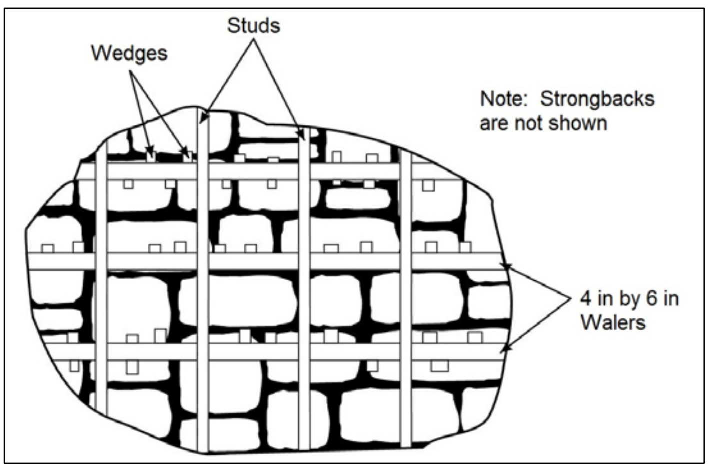


Figure 500 – 41: Wedge Method of Securing Stone Masonry Faces for Concrete Placement

Alternatively, some designers or Contractors opt to pour the concrete first, allowing for the stone facing to be constructed and secured to the concrete afterwards through the use of dovetail anchors that are inserted and twisted into slotted strips embedded vertically in the concrete.

- Chamfer Strips

The use of chamfer strips at the exposed corners and edges of finished work is generally required to provide an eye–appealing line on the concrete. Almost without exception, chamfer strips should be used on the front three sides of bridge seats even though forms are cut to grade. Their use here protects the edges from breaking or chipping. The District Construction Engineer should be consulted regarding the use of chamfer strips on the project.

Figure 500 – 42: Concrete Chamfer Strip

F. Falsework

Falsework is the term that describes the load bearing portions of forms where the form supports the concrete vertically rather than from the side. Cantilevered portions of pier caps, decks, abutments and wings, and slabs of rigid frame bridges and culverts are some of the placements that require careful consideration in the design of falsework. The falsework must be sufficient to withstand the placement of concrete loads without deformation.

Figure 500 – 43: Installing Concrete Formwork Falsework

Falsework plan submissions to the Construction office are required of the Contractor three weeks prior to the beginning of concrete operations so that there is sufficient time for review and documentation. In this manner, the Contract Administrator can inspect construction of the falsework forms for compliance with the accepted plan. Falsework on decks usually necessitates timber being used around the steel girders and the abutment. The Contract Administrator must inspect this area to ensure that no timber will bind between the steel and the backwall should the bridge expand due to changes in temperature. Falsework timber should be inspected for strength deficiencies such as cross grain, large knots, cracks, and dry rot.

The Contractor shall follow these requirements for submitting bridge falsework plans for approval and documentation:

- The Contractor shall submit five sets of all falsework plans and calculations to the Construction Bureau directly or through the Contract Administrator.
- The Contractor's falsework plans shall be stamped by a New Hampshire Registered Professional Engineer per the Specifications.
- All falsework drawings shall be neatly drawn to a suitable scale, and all details shall be clearly shown. Lumber species, grade, and allowable basic stresses in bending, horizontal shear, compression perpendicular and parallel to grain, and the modulus of elasticity shall be noted on the submitted plans. Unless indicated otherwise, the stock shall be assumed as dressed lumber.
- For construction purposes, the basic allowable stresses shall be increased by 25%.
- Computation for deflection shall be made in order to have the resulting shape match the design with satisfactory tolerance. Live load may be omitted from wood falsework deflection computations. For spans of 12 ft or less, wood falsework shall have a resulting deflection of less than 1/600 of the span.
- Falsework resting on the original ground or on compacted fill shall be supported by continuous sills that are suitably spliced. For wood falsework, the soil shall not be expected to support more than 2000 lbs/ft^2.
- Special attention shall be given to the falsework's lateral bracing in all planes of movement.
- The use of shimming devices, such as hardwood wedges, for vertical control of settlement shall be clearly shown.
- Special proprietary devices such as jacks, hangers, and steel scaffolding shall be referenced to a standard catalog, or a catalog may be supplied with the submission. Such catalogs shall supply all the required details and the allowable loading recommended for the device in question.
- Any exterior beam overhang supports (brackets) that induce excessive torsion into the beam shall be prohibited.

G. Expediting Delivery

The Contractor and the Contract Administrator should discuss all aspects of delivering and placing the concrete before it arrives on the job to ensure that the work area is properly organized. Access for the mixer trucks should be given consideration, and chutes and tremies must be properly arranged for an efficient procedure. At this time, the Contractor should assure the Contract Administrator that the necessary personnel are available with standby vibrators and generators should there be any breakdowns.

Note: The Contractor should be advised that the use of aluminum chutes, tremies, and finishing tools are prohibited.

H. Weather Conditions

Rainy weather is another variable that the Contract Administrator has to contend with once the forms are up, checked, and the Contractor is ready for the placement. Concrete placements should never be started in the rain unless adequate protection can be provided. In addition to rainwater being added to the concrete and affecting the water–cement ratio, rain spots will appear on surfaces that are uncovered. Concrete shall not be placed when the atmospheric temperature is below 35°F without provisions for external heat with housing or insulated forms and blankets.

I. Concrete Plant Inspectors

The Concrete Plant Inspector is the Contract Administrator's representative at the batching plant whose main responsibility is inspecting the preparation of the concrete mix. The Concrete Plant Inspector may be one of the assigned project personnel, an Inspector from another project, or a Department–hired Consultant Inspector.

Should the Contract Administrator need a Concrete Plant Inspector to cover a placement aside from the personnel assigned to the project, arrangements should be made with the current low bidder consultant testing company as directed by the Materials and Research Bureau. These inspection companies are generally short–staffed and busy, and require sufficient lead time to provide inspection staff. The information required on the Concrete Plant Inspector's work sheet (also known as the "call" sheet) should be filled out in full and relayed to the Consultant Inspector's office when requesting coverage.

J. Concrete Plant Testing

Before a concrete plant's products may be approved for use on NHDOT projects, NHDOT concrete laboratory personnel from the Materials and Research Bureau will have made thorough on–site inspections of the plant. The Concrete Plant Inspector must perform many spot checks in order to ensure that concrete of a consistent quality is being produced. Upon receiving the concrete plant work sheet (call sheet), the Concrete Plant Inspector should review the information for clarity and completeness. If any issues arise, the contact person listed on the concrete plant work sheet (call sheet) should be immediately called. The Inspector should not wait until the concrete plant person is on site to inform them about a previously discovered problem with the mix, as this may delay the entire placement operation.

The Concrete Plant Inspector must arrive at the plant in sufficient time to perform tests and inspections before the first load is ready to be batched out. The time anticipated for the first truck to arrive on site is indicated on the concrete plant work sheet (call sheet). This time is critical and should not have to be pushed back due to delays in plant inspection.

The Inspector, after introductions are made to the plant batch person, should verify that there are sufficient cement and any other necessary materials on hand to cover the planned concrete placement. The information entered by the plant batch person and the concrete plant work sheet (call sheet) should match. The Inspector may be requested to sign the batch plant's logbook, and if so, they should post a copy of the test results inside it before leaving for the day.

Next, the Concrete Plant Inspector should gather aggregate samples for the gradation and fineness modulus (FM) tests. Moisture content calculations will be used to compute the actual concrete mix by adjusting the design weights shown on the concrete plant work sheet (call sheet). The process for performing these calculations is found in the following subsections.

K. Conducting Concrete Plant Moisture Content Calculations

There are several theoretical factors that must be related to moisture content testing. Batch weights used in the design of any concrete mixture are based upon saturated surface dry (SSD) aggregate weights. However, equipment required at each concrete plant, by specification, and the drying procedure used cannot compare the actual sample with the SSD standard. Therefore, a correction factor known as the absorption factor must be used.

This factor is included in the mix design provided by the Bureau of Materials and Research prior to the start of non–QC/QA concrete work on any project. The information will be forwarded to the Concrete Plant Inspector on the concrete plant work sheet (call sheet). If the absorption factor is unknown or unavailable, the Inspector should use a value of 1% and make note of this assumption on the field test report.

The Inspector should choose a representative sample from the bins or stockpiles to perform the tests. Obtaining a representative sample is extremely important to the outcome of the concrete mix test results. Samples should not be taken by the Inspector until the all of the conditions are right to collect proper representative samples.

Usually, much of the excess water in the aggregate storage bins will sink to the bottom of during the night. If the plant has not been operating prior to the Inspector's arrival, the batch person should drop some aggregate from the storage bin and then the plant personnel should mix the pile and build a sample platform. The Inspector should take representative samples from this material.

Once batching operations begin, the Inspector shall closely monitor operations to verify that the material being fed to the plant is of the same nature as the sampled material.

Damp aggregates will normally have about 3% moisture in the sand and 1% in the stone. Under certain conditions, very wet sand may have 6% to 10% moisture and the stone may hold 3% to 4%. Therefore, it is very important to know how much free water must be compensated for in order not to exceed the proper water/cement ratio.

First, test the sand aggregate for moisture content. Weigh out a sample of about ¾ lb of damp sand, recording the "wet weight." Dry the sand sample over a stove or other type of heating device and then weigh the sample again, recording the "dry weight."

Calculate the net weight of the wet and dry sand samples, the weight of water in the sample, and the moisture content of the sample.

Given:

T_{WET} = Total Weight of Wet Sand Sample = 345 g

T_{DRY} = Total Weight of Dry Sand Sample = 332 g

W_{PAN} = Tare Weight of Pan = 45 g

Find:

N_{WET} = Net Weight of Wet Sand Sample

N_{DRY} = Net Weight of Dry Sand Sample

W_W = Weight of Water in Sand Sample

MC = Moisture Content of Sand Sample

Calculate:

$N_{WET} = T_{WET} – W_{PAN}$

$N_{WET} = 345 – 45 = 300$ g

Therefore, the net weight of the wet sand sample is 300 g,

and

$N_{DRY} = T_{DRY} – W_{PAN}$

$N_{DRY} = 332 – 45 = 287$ g

Therefore, the net weight of the dry sand sample is 287 g,

and

$W_W = N_{WET} – N_{DRY}$

$W_W = 300 – 287 = 13$ g

Therefore, the weight of water in the sand sample is 13 g,

and

$MC = W_W / N_{DRY}$

$MC = 13 / 287 = 0.0453 = 4.53\%$

Therefore, the moisture content of the sand sample is 4.53%.

The sand sample lost 13 g of water and is now absolutely dry. The mix design gives SSD values for each aggregate and the absorption factors.

L. Other Concrete Plant Testing Procedures and Calculations

Consider a mix design that requires 1,130 lbs of sand (sampled in the previous subsection) with an absorption factor of 0.6%. An absorption factor of 0.6% means that the aggregate will soak up a weight of water equal to 0.6% of its dry weight. The portion of the moisture content that remains once the particles are saturated will give an indication of the "free water" in the material. This free water is the water available in the aggregate stockpile for hydration with the cement.

The amount of free water in the sand is calculated as follows:

Given:

MC = Moisture Content of Sand

AF = Absorption Factor of Sand

Find:

W_{PF} = Percentage of Free Water in Sand

Calculate:

$MC – AF = W_{PF}$

$4.53 – 0.60 = 3.93\% \approx 4\%$

Therefore, the percentage of free water in the sand is 4%.

Considering the mix design again, if 1,130 lbs of sand is weighed out, the actual amount with at 4% free water is calculated as follows:

Given:

S_{TOTAL} = Total Sand Amount Required for Mix = 1,130 lbs

W_{PF} = 4.0% or 0.04

Find:

W_{FREE} = Weight of Free Water in Sand

S_{NET} = Net Sand Amount Required for Mix

Calculate:

$W_{FREE} = S_{TOTAL} \times 0.04$

$W_{FREE} = 1{,}130 \times 0.04 = 45.2$ lbs

Therefore, 1,130 lbs of wet sand with 4% free water contains 45 lbs of water.

and

$S_{NET} = S_{TOTAL} – W_{FREE}$

$S_{NET} = 1{,}130 – 45 = 1{,}085$ lbs

Therefore, the net weight of the sand is 1,085 lbs. Since the mix requires a total of 1,130 lbs of dry sand, 45 lbs of sand should be added to sand to bring the total amount of sand used in the mix to 1,130 lbs. This method is not absolutely accurate, but the difference in the amount of sand to use in the mix must be accounted for.

The procedure for testing and calculation of the moisture content of the coarse aggregate is similar. The absorption correction factor is likely different and would need to be used.

The amount of water to be added at the plant is not stated on the concrete plant work sheet (call sheet) for a reason. The Concrete Plant Inspector and the Contract Administrator are not responsible for telling concrete plant personnel the amount of water to be used in the mix. The Concrete Plant Inspector provides the batch person with the moisture content of aggregates from the tests and the expected slump to be used on the project, but it is the batch person who decides how much water to add.

The batch person knows the theoretical amount of water for each cubic yard and can estimate the amount to hold back for mixing. The amount of water added to the mix load should be written on the delivery slip. At the work site, the transit mix driver may add water to provide for the specified slump, with this also noted by field personnel on the delivery slip.

By not specifying the amount of water to be added to the mix, the Inspector is not responsible for wet loads. The water amounts noted on the delivery slips (the aggregate free water, the water added at the plant, and the water added on the work site) may be used to figure the water to cement ratio.

The next step is to determine the fineness modulus (FM) of the sand aggregate. According to the *Whole Building Design Guide,* from the National Institute of Building Science, the standard definition of fineness modulus is as follows:

"[The Fineness Modulus is] an empirical factor obtained by adding the total percentages of a sample of the aggregate retained on each of a specified series of sieves, and dividing the sum by 100."

Note: The sieve sizes used are #100 (150 μm), #50 (300 μm), #30 (600 μm), #16, #8, and #4, and ⅜ in, ¾ in, 1½ in, and larger, increasing in the ratio of 2:1.

According to the Specifications, the FM factor of sand should be between the values of 2.5 and 3.1. The FM factor used in designing the mix will be shown on the concrete mix design sheet provided by the Materials and Research Bureau. If the FM factor varies more than 0.2+/– from that used in the design, a redesign of the mix may be necessary. Calculations to determine the fineness modulus factors are shown in the following subsections.

The coarse aggregates are run through the sieves for gradations. To help the Concrete Plant Inspector in recording information on the field test reports and transit mix delivery slips, the concrete plant work sheet (call sheet) should be used. All tests must be completed prior to the batching of the first truckload. A completed gradation test report should be delivered to the onsite project personnel with the first delivery slip.

Testing procedures for fine and coarse aggregate used in concrete; and samples of concrete plant work sheets, gradation test reports and concrete batch and delivery records are shown in the following subsections.

M. Testing Fine Aggregate Used in Portland Cement Concrete

<u>Scope</u>: Fine aggregates used in the production of Portland cement concrete have a definite effect on the design of a concrete mix. Besides influencing the amount of water required to make a workable mix, the mechanical composition of the aggregate is of concern since it affects the strength of the mix. In general terms, a fine aggregate with a high FM is

coarser than one with a lower FM. The purpose of this test is to determine the fineness modulus of the fine aggregates used in Portland cement concrete.

Apparatus: Along with a representative sample of the fine aggregate, the following equipment is required to conduct FM testing for fine aggregate:

- A scale with a tolerance of +/–0.1 grams
- Eight nested sieves as follows: ⅜ in, #4, #8, #16, #30, #50, #100, and #200 (and a pan)
- Mechanical shaker

Procedure: Use the following procedure to conduct the FM testing for fine aggregate:

1. Dry about 700 grams of fine aggregate.
2. Arrange the sieves in descending order so that the ⅜ in sieve is on top, the #4 sieve is next and so forth down to the #200 sieve, which should be resting on the pan.
3. When the fine aggregate has cooled, weigh out a sample of approximately 500 g of material.
4. After recording the weight, place the sample of material in the ⅜ in sieve, cover, and clamp the entire nest of sieves into the mechanical shaker.
5. Shake for 3 to 5 minutes.
6. Remove the ⅜ in sieve and weigh the material retained on it. Record this weight.
7. Next add the material retained on the #4 sieve to the material previously weighed and record this cumulative weight. Continue this process for the remaining sieves, recording the cumulative weight for each sieve.
8. Calculate the percent of material retained for each sieve by using the cumulative weight recorded, divided by the original known weight of the sample.

The FM is calculated by dividing the summation of percents retained on all sieves except the #200 sieve by 100.

Given:

FM = Fineness Modulus

FM = [Sum of Retained Percentages] / 100

Note: The determination of the percent passing, required for filling out the gradation test report (otherwise known as the "gray sheet"), is found by simply subtracting the percent retained from 100.

N. Testing Coarse Aggregate Used in Portland Cement Concrete

Scope: In addition to the fine aggregate test described above, the Concrete Plant Inspector is also required to run a gradation on the coarse aggregate. The procedure is similar to that used to determine the gradation of the fine aggregate.

Apparatus: Along with a representative sample of the coarse aggregate, the following equipment is required to conduct FM testing:

- A scale with a tolerance of +/–0.01 lbs
- The following sieves: 1½ in, 1 in, ¾ in, ½ in, ⅜ in, #4, #8, #16,and #50; the #16 and #50 sieves are only used for an overlay concrete mix design
- Mechanical shaker

Procedure: Use the following procedure to conduct the FM testing for coarse aggregate:

1. The gradation testing of the coarse aggregates is conducted in a similar fashion to that of fine aggregates. Calculate the percent passing for each sieve that is required for the coarse aggregate sample. Refer to *Subsection 520.2.2.2.2.2 Required Grading: Table 3 – Coarse Aggregate* of the Standard Specifications for more information. Where the storage of the coarse aggregate is in two or more bins, a sample for each bin is taken and a gradation test is performed on the individual sample. It is then necessary to combine the results of the individual tests so a comparison with the Specifications can be made.

2. In combining the test results, verify the proportion of aggregates in the concrete mix design. The percentage of each size of coarse aggregate can be determined from the Concrete Plant Inspector's work sheet (call sheet). For instance, the example concrete plant work sheet (call sheet) on the following pages shows approximately 60% of #4 – ⅜ in stone (106.9 / 1780) and 40% of ⅜ – ¾ in stone (711 / 1780).

3. In this case, multiply the gradation test result for the #4–⅜ in stone by 0.60 and the result for the ⅜ – ¾ in stone by 0.40. Add these results, which represent the proportioned gradation, and compare this value with the Specifications. If in doubt as to the proportioning, consult the concrete mix design sheet from the Bureau of Materials and Research and use the corresponding percents as the multiplier.

The following tables show a sample concrete plant work sheet.

State of New Hampshire / Department of Transportation					
Bureau of Construction					
Concrete Plant Work Sheet – Call Sheet (1)					
Project	Keene 12345	Date	September 15, 2015		
Contract Administrator	Ronald Tanner	Project Phone	669–5554		
		Pager Number	662–0132		
Plant Location	Whitcomb – Keene	Home Phone	946–7381		
		Cell Phone	419–1178		
Bridge Location	082/130 Route 12 – B&M RR	Concrete Class	AA	Item No.	520.01
Section being poured	Abutment ‘A’ Backwall	Estimated yd^3	30		
Absorption Factor (fine aggregate)	0.746%	Design FM	2.65		
Absorption Factor (coarse aggregate)	0.438%	Slump:	2–4 inches		
No. of bag mix:	7		Water/Cement:	0.444	
Heat Aggregate ☐	Cool Aggregate ☐	Hot Water ☐	Pour Start Time:	7:00 AM On Site	

State of New Hampshire / Department of Transportation								
Bureau of Construction								
Concrete Plant Work Sheet – Call Sheet (2)								
A	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**
Aggreg. Size	1 yd^3 dry weight (lbs)	% free water (minus absorp. factor)	Gallons of free water per yd^3	1 yd^3 wet weight (lbs)	(8) yd^3 wet	Gallons of free water (8) yd^3	(3) Drop weights (lbs)	Acum. Drop weights (if used) (lbs)
		(÷100)	C x B = D	(C x B) + B = E	E × (8) = F	D × (8) = G	F / (3) = H	
Sand	1,185	0.036	5.12	1227.66	9821.3	41.0	3273.8	3273.8
#4– ⅜ in	1,069	0.01	1.28	1079.69	8637.5	10.2	2879.2	6153.0
⅜– ¾ in	711	0.005	3.56	714.56	5716.5	28.5	1905.5	8058.5
Cement	329	lbs/yd^3	× ( ) yd^3 =		2632		877	
Pozzo.	329	lbs/yd^3	× ( ) yd^3 =		2632		877	
Air Entrain.	2.6	oz/yd^3	× ( ) yd^3 =		20.8			
Mid–range	30	oz/100 lb	× (C+P)/100 × ( ) yd^3 =		1579.2			
HRWR		oz/100 lb	× (C+P)/100 × ( ) yd^3 =					

State of New Hampshire / Department of Transportation								
Bureau of Construction								
Concrete Plant Work Sheet – Call Sheet (3)								
A	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**
Aggreg. Size	1 yd^3 dry weight (lbs)	% free water (minus absorp. factor)	Gallons of free water per yd^3	1 yd^3 wet weight (lbs)	(8) yd^3 wet	Gallons of free water (8) yd^3	(3) Drop weights (lbs)	Acum. Drop weights (if used) (lbs)
	E x ( ) = F	D x ( ) = G	F / ( ) = H	(lbs)	E x ( ) = F	D x ( ) = G	F/( ) = H	
Sand								
#4– ⅜ in								
⅜– ¾ in								
Cement								
Pozzo.								
Air Entrain.								
Mid–range								
HRWR								

Note: The information to be added in the cells within the dashed lines above is to be provided by the Contract Administrator. The remainder of the worksheet is to be filled out by the Concrete Plant Inspector.

State of New Hampshire / Department of Transportation							
Bureau of Construction							
Concrete Plant Work Sheet – Call Sheet (4)							
⅜ in	Weight (lbs)	Tare (lbs)	Net Weight (lbs)				
Total Weight	15.40	1.10	14.30	% Retained	% Passing	% Stone Mix	%
½ in	1.1	1.1	0	0.0	100.0	60.0	60.0
⅜ in	7.15	1.1	6.05.	42.3	57.7	60.0	34.6
#4	14.0	1.1	12.90	90.2	9.8	60.0	5.9
#8	14.97	1.1	13.87	97.0	3.0	60.0	1.8
#16	15.06	1.1	13.96	97.6	2.4	60.0	1.4
#50	15.13	1.1	14.03.	98.1	1.9	60.0	1.1
¾ in	Weight (lbs)	Tare (lbs)	Net Weight (lbs)				
Total Weight	26.45	1.1	25.35	% Retained	% Passing	% Stone Mix	%
1 in	1.1	1.1	0	0.0	100.0	40.0	40.0
¾ in	7.41	1.1	6.31	24.9	75.1	40.0	30.0
⅜ in	18.92	1.1	17.82	70.3	29.7	40.0	11.9
#4	24.27	1.1	23.17	91.4	8.6	40.0	3.4
#8	26.15	1.1	25.05	98.8	1.2	40.0	0.5

State of New Hampshire / Department of Transportation							
Bureau of Construction							
Concrete Plant Work Sheet – Call Sheet (5)							
1½ in	Weight (lbs)	Tare (lbs)	Net Weight (lbs)				
Total Weight				% Retained	% Passing	% Stone Mix	%
2 in							
1½ in							
¾ in							
⅜ in							
#4							

	⅜ in	¾ in	$1^1/_2$ in	Total	Spec	Moisture				
2 in							Total Wgt. (lbs)	Tare (lbs)	Net Wgt. (lbs)	
$1^1/_2$ in						Wet Sand	474.5	20.0	454.5	A
1 in	60	40		100	100	Dry Sand	455.6	20.0	435.6	B
¾ in	60	30		90	90–100		Wet – Dry		18.9	C
⅜ in	34.6	11.9		46.5	20–55					
#4	5.9	3.4		9.3	0–10	C = 18.9 = 4.34%		0.746% = 3.60%		
#8						B = 435.6 (Absorption)				

State of New Hampshire / Department of Transportation						
Bureau of Construction						
Concrete Plant Work Sheet – Call Sheet (6)						
Sand	**Weight (g)**	**Tare (g)**	**Net Weight (g)**			
Total Weight	455.6	20.0	435.6	% Retained	% Passing	Spec
#4	24.4	20.0	4.4	1.0	99.0	95–100
#8	74.4	20.0	54.0	12.4	87.6	
#16	148.5	20.0	128.5	29.5	70.5	45–80
#30	250.0	20.0	230.0	528	47.2	
#50	355.4	20.0	335.4	77.0	23.0	10–30
#100	430.8	20.0	410.8	94.3	5.7	2–10
FM : →				2.67		
#200		20		97.8	2.2	0–3
Tested By			Michael M. Peters			

State of New Hampshire / Department of Transportation								
Bureau of Construction								
Concrete Gradation Test Report (1)								
Project:	Keene 12345				Field Test No.:	MMP032015–01		
Type of Material:	Concrete Aggregate				Date Reported:	September 15, 2015		
Reported by:	Michael M. Peters				Received by Lab:			
Report to:	Project Files	☒	Lab	☐	Contractor:		☐	
Sampled Date:	9/15/2015		At (town):		Keene, NH			
Source of Material			Whitcomb Materials					
Sample From:	Stockpiles	☐	Pit	☐	Roadway	☐	Sta.:	
Quantity (Represented or Estimate):			$30\ yd^3$ +/–					
Purpose / Location:			Abut A Backwall			Item No.:	520.01	
Tested	Gradation	☒	F.M.	☒	% Moisture	☒	Date:	9/15/2015

Sieve	Size ⅜ – 60 %	Size ¾ – 40 %	Size %	Combined Results	Required Spec.
	% Passing	% Passing	% Passing		
	Coarse Aggregates and Gravels				
6 in					
3½ in					
3 in					
2½ in					
2 in					
1½ in					
1¼ in					
1 in	60.0	40.0		90.0	100
¾ in	60.0	30.0		90.0	90 – 100
½ in					
⅜ in	34.6	11.9		46.4	20 – 55
#4	5.9	3.4		9.3	0 – 10

State of New Hampshire / Department of Transportation						
Bureau of Construction						
Concrete Gradation Test Report (2)						
Project:	Keene 12345			Field Test No.:	MMP032015–01	
Type of Material:	Concrete Aggregate			Date Reported:	September 15, 2015	
Reported by:	Michael M. Peters			Received by Lab:		
Report to:	Project Files ☒	Lab ☐		Contractor:	☐	
Sampled Date:	September 15, 2015	At (town):		Keene, NH		

	Size ⅜ – 60 %	Size ¾ – 40 %	Size %	Combined Results	Required Spec.
Sieve	% Passing	% Passing	% Passing		
	Fine Aggregates and Sands (Washed ☐)				
#4	99.0				95 – 100
#8	87.6				
#16	70.5				45 – 80
#30	47.2				
#50	23.0				10 – 30
#100	5.7				2 – 10
F.M.	2.67				2.65±0.2
#200 in Sand	2.2				0 – 3
% Moisture	3.6				
Remarks:					

Meets requirements for:	Item 520.01 Class AA Concrete	See reverse	☐
	Tested by:	Michael M. Peters	
		Signature:	

When the Inspector has completed the above gradations, the following form needs to be filled out and sent to the project on the first load along with this batch and delivery record. This is the placing Inspector's opportunity to verify that all the aggregate gradations meet Specifications. One copy of each gradation test shall be given to the Contract Administrator for their records and also be maintained at the concrete plant in a ringed binder as a record of the gradation history.

With all tests completed, the Inspector should then prepare the Concrete Batch and Delivery Record slips and provide the batcher with the ingredient weights. Continuing the example begun above, using the Concrete Plant Work Sheet and the test results, the delivery slip would be filled out as in the following example.

<table>
<tr><th colspan="10">State of New Hampshire / Department of Transportation</th></tr>
<tr><th colspan="10">Construction Bureau</th></tr>
<tr><th colspan="10">Concrete Batch and Delivery Record</th></tr>
<tr><td>Project:</td><td>Keene 12345</td><td>Date</td><td>9/15/15</td><td></td><td>Weather:</td><td>Fair</td><td>Air Temp.:</td><td colspan="2">42ºF</td></tr>
<tr><td>Truck #:</td><td>23</td><td>Batch #:</td><td>1</td><td></td><td colspan="2">Size (A): 8</td><td colspan="3">Sub–Total: 8 yd^3</td></tr>
<tr><td colspan="2">Mix Weights on Batch #: 1</td><td colspan="2">Total Delivered Water (B): 244 gal</td><td></td><td colspan="2">Job Arrival</td><td>7:05 AM</td><td colspan="2"></td></tr>
<tr><td colspan="2">Total Delivered Weight (C): 30,795 lbs</td><td colspan="2">Batch Time: 6:25 AM</td><td></td><td colspan="2">Start Mixing</td><td>7:12 AM</td><td colspan="2">RPM: 17</td></tr>
<tr><td colspan="2">Plant Inspector:</td><td colspan="2">Employee #:</td><td></td><td colspan="2">Start Discharge</td><td>7:18 AM</td><td colspan="2"></td></tr>
<tr><td colspan="2">Michael M. Peters</td><td colspan="2">72665</td><td></td><td colspan="2">Finish Discharge</td><td>7:42 AM</td><td colspan="2"></td></tr>
<tr><td colspan="4">Complete the following for the first load of each truck size or when the mix changes:</td><td></td><td colspan="5">Water added on site: 20 gals × 8.33 = 167 lbs (D)</td></tr>
<tr><td>Source:</td><td>Whitcomb – Keene</td><td>Class:</td><td>AA</td><td></td><td colspan="5">Total Delivered (B): 244 gals</td></tr>
<tr><td>Location of pour:</td><td colspan="3">Abut A Backwall</td><td></td><td colspan="5">Total Water (F): 264 gals / Size (A) 8 = 29.25 gal/yd^3</td></tr>
<tr><td>Scale Inspection Date:</td><td>3/1/2015</td><td>Water Temp.</td><td>120°F</td><td></td><td colspan="5"></td></tr>
<tr><td>Moisture:</td><td>Free Water:</td><td colspan="2">9,280 lbs</td><td></td><td>Cylinder Numbers:</td><td>040–042</td><td>2¾ in</td><td>Air</td><td>6.2%</td></tr>
</table>

State of New Hampshire / Department of Transportation								
Construction Bureau								
Concrete Batch and Delivery Record (2)								
Project:	Keene 12345	Date	9/15/15	Weather:	Fair	Air Temp.:	42 ºF	
Truck #:	23	Batch #:	1	Size (A): 8		Sub–Total: 8 yd^3		
	Moisture:	Free Water:	9,280 lbs	Cylinder Numbers:	040–042	2¾ in	Air	6.2%
Sand	3.6%	41 gals	8,635 lbs	Unit Weight:				
#4–⅜ in	1.0%	8 gals						
⅜ – ¾ in	0.5%	29 gals	5,710 lbs	Weight: 43.35 lbs				
#4 – 1½ in	%			– Bucket	7.45	Conc. Weight: 35.90 = 143.60 lbs/ft^3 (E)		
Water added at Plant:		164 gals × 8.33 = 1,366 lbs		Conc. Wt. 35.90		Volume: 0.25		
Total Delivered Water		244 gals (B)		Yield: (C + D) = (30,795+167) = 27.10 ft^3/yd^3				
Cement Wt.: 2,632 + Pozzolans = 5,264 lbs G)				(E x A)	(143.60 × 8)			
Total Delivered Weight (C)		30,795 lbs		W/C Ratio: F × 8.33 = 2,199 = 0.418			Microwave: 0.423	
Air Entrainment		Type:	Darex II	At plant	21 oz		(G) 5,264	
Midrange		Type:	WRDA 65	At plant	1,579 oz			
HRWR		Type:		At Plant:				
		Type:						
Concrete Placing Inspector				Employee Number				
Ronald Tanner				56627				

Note: The left side of this Concrete Batch and Delivery Report should be completed by the Concrete Plant Inspector for the first load of each truck size or when the mix changes due to moisture content variations, etc. However, only the top portion needs to be completed thereafter.

Note: The time and amount of water added to the mix should be completed on every slip by the Concrete Placing Inspector, with other portions to be completed at appropriate intervals.

O. Concrete Plant Batching

Immediately prior to the batching of materials, the Inspector shall verify that all trucks scheduled to deliver to the project are back spun and display a valid inspection sticker issued by the Materials and Research Bureau. The Inspector should continue to oversee the entire plant and stockpile operation in order to spot any irregularities that may affect the consistent quality of the concrete product.

Batching plants are classified in three categories: manual, semi–automatic, and automatic. Only a few manual plants are still in existence and approved to be used for state work, but most plants in operation today are automatic or semi–automatic. In a manual batching plant, the batcher controls all the functions necessary to complete the weighing and loading in the manual plant. This type of plant is subject to human error in all parts of its operations and the Inspector should closely monitor its batching activities. A semi–automatic plant is governed by controls that are actuated in a certain sequence to complete the batching cycle. An automatic plant is considered one where the complete batching cycle is set in motion by a control button that may be located remotely from the plant.

Figure 500 – 44: Concrete Batching Plant

The batching materials scales are inspected, calibrated, and approved annually by authorized Department personnel and may be identified by a seal with the date of last inspection. The Inspector must record this scale inspection date on the delivery slip. The Concrete Plant Inspector also must check the scale system.

A modern batching materials scale has a series of compound levers arranged to balance a load of thousands of pounds at one end with a calibrated balancing weight of comparatively few pounds at the other end. The ratio of this lever system is on the order of 100:1 or more. The levers pivot on knife–edges bearing on flat blocks. Both the knife–edges and bearing blocks are fabricated from very hard, but brittle, steel. Good practice calls for daily visual checks of all pivot points and constant observation of the dial or telltale indicator during batching operations. Erratic movement of these would suggest binding in the lever mechanism.

The possibility of cement shortages due to cement sticking in the weigh hopper has been well recognized by experienced construction personnel for some time. Another problem is not so well known, and it involves the use of air pressure for the movement of cement to the weighing hoppers during delivery. There may be still be older batching bins in use that are designed for air pressures of about 15 psi using a 3 to 4 in diameter vent. Cement trucks are now using pressures of up to 44 psi and unless the vents are increased to 8 to 10 in, the increased air pressure may give a false reading on the scale.

A quick check to verify if this is occurring can be made by filling the cement hoppers and allowing them to stand for 10 minutes. If the scale indicator moves during this 10 minute interval it can be assumed that air pressure is affecting the scale reading and that additional venting is needed. The Inspector should then request that cement delivery cease until the batching of cement into the load has been completed.

Water metering or measuring apparatus should be checked periodically to see that only the required amount is batched. Leaking valves can result in extra water entering the mixer. Weigh bins should be checked frequently to see that they are discharging completely and that material is not hanging up in the corners.

The addition of the air–entraining agent should be checked to make sure that the intended amount actually gets into the batch and should be dispensed so that it goes into the sand and not on the stone. Other admixtures, such as water reducers and retarders, should also be carefully monitored. The dosages of these may be altered during the placement based upon on–site test results so the Inspector should ensure that the batcher is notified of any changes.

P. Concrete Plant Cement Sampling

The Concrete Plant Inspector's final requirement is to submit a sample can of cement along with the cement test report or mill test report should to the lab at the Materials and Research Bureau. This may be accomplished during one of the loading batches by sending the sample with one of the transit mix drivers to the job site to be given to the on–site state Inspector. If the Concrete Plant Inspector returns to the same plant the next day, or on a succeeding day, and no new shipment of cement has been received, the new sample may be tagged bearing the same information regarding shipment number, supplier, etc. This information should then be copied from the plant's copy of the mill test report.

The Concrete Plant Inspector has an important function and must do everything possible to expedite changes and adjustments in the mix requested by the Contract Administrator during the pour. Any changes in the mix should be noted on the back of the delivery slips for future reference and documentation. And, finally, before leaving the batch plant, testing equipment should be cleaned and put away so that it will be ready to use by the next Concrete Plant Inspector and not left as an unsightly mess for the batcher.

Note: The Concrete Plant Inspector should not leave the plant until they have verified that all the concrete that the project requires has been batched, regardless of the amount originally ordered.

Q. Placing Concrete

The Contract Administrator should encourage the Contractor to have a worker meet and guide the concrete trucks when they arrive at the work site. The Pouring Inspector should inform the driver of the specification limits for slump. During the first round of truck deliveries, the Placement Inspector should work closely with the delivery personnel until drivers become familiar with the slump and consistency requirements.

The Placement Inspector should complete the field tests for slump and air entrainment before any concrete placement operations have commenced, but all subsequent tests should be taken in the middle of the load. The Placement Inspector should check the initial load for proper placement and vibration to establish proper procedures for the remainder of the pour.

Figure 500 – 45: Placement Inspector Conducting Concrete Sample Testing

Previously poured concrete at construction joints should be wetted along with the forms prior to commencing placement; however, water should not stand in the bottom of the forms. This prior wetting becomes very important in warm weather when backing up granite curb so that the granite does not draw water out of the concrete. For proper sampling procedures, refer to *Division 700 Method of Making and Curing Concrete Cylinders.*

- Underwater Concrete Placement

Placement of concrete under water must be accomplished so that cement will not be washed out of the mix by using a rigid steel tube called a tremie or by using a pump truck with rigid rather than flexible hose at the discharge end. At no time shall concrete be allowed to fall through water. The lower end of the tremie/pump must always be kept embedded in previously placed concrete. The tremie/pump can be moved laterally like a pendulum by first charging it with concrete, moving the top of the tremie/pump to the new location, and lifting and discharging the concrete.

It is important to monitor the elevation of the bottom of the tube to ensure that it is not lifted out of the concrete during the discharge lift. The depth of the tube can be easily monitored by marking elevations on the tremie/pump. During the discharge lift, the bottom of the tube will swing to the new location in the concrete.

It may be necessary to remove the tremie/pump from the concrete in order to move over a brace to another part of a cofferdam. Upon reentry, water can enter the tube. Therefore, a reseal must be made and care must be used not to flush water through previously placed concrete causing segregation. It is possible to close off the end of some tremie/pump hoses to water by duct taping a piece of polyethylene over the end of the tube. The pressure of the discharging concrete will break the tape after it has been placed in the previous layer of concrete. This allows the tremie/pump to continue concrete discharge without any flushing action.

Basketballs are also commonly used to block the hose during reentry into the water. Concrete placed under water must not be vibrated or agitated. Cofferdams in running water must be tight enough to prevent loss of cement due to current action.

Competent supervision is of primary importance in any operation as complicated as an underwater concrete tremie placement. Concrete should be deposited under water only under the immediate supervision of project personnel. A vital part of the supervision lies in checking out preparations for a tremie operation. Elevation references should be set at several points around the cofferdam. A probe, level rod or sounding line should be on hand to check concrete elevations during placement.

- Footing Concrete Placement

  No footing concrete should be placed until the foundation has been inspected for stability and load bearing properties; and has been approved by the Contract Administrator. Likewise, no concrete should be placed in any portion of the structure until field personnel have had sufficient time to fully inspect the reinforcement, forms, falsework, mud sills, and tell–tales and found these elements to be satisfactory.

- Columns or Thin Wall Placement

When placing concrete in columns or thin walls where a comparatively small volume is required, it is quite easy to overload the forms with a fluid head of unset concrete. For example, if the assumption is made that the initial set time of portland cement is between 3 and 4 hours at 70°F and the forms are calculated to be capable of holding a 10 ft depth of unset concrete, then it would be dangerous to allow a pour build–up greater than about 7 ft per hour at 70°F. This build–up should be reduced to about 5 ft per hour at 50°F.

- Segregation Prevention

Proposed placement methods and equipment should be checked in advance of work to verify their proper planning and operation. Mix segregation may occur because of poor placement practice. When concrete is placed from a chute, when permitted, the coarse aggregate could separate out and form a rock heap almost devoid of mortar, while the mortar runs off separately. Concrete placed carelessly in wall forms may end up segregated by the action of coarse aggregate particles striking form ties or reinforcing steel.

Specifications require that concrete chutes should be of metal or be metal–lined, but not with aluminum. Specifications also require that concrete be placed in a wall or column form by means of a metal or rubber elephant trunk to eliminate dropping the concrete more than 5 ft.

- Curing

Concrete surfaces should be checked frequently during the curing period to see that they are always wet. When polyethylene plastic sheeting is used to cover slabs for curing, the sheeting should be lifted periodically in places to ascertain that the concrete surface actually is wet. Presence of moisture on the underside of the sheeting does not necessarily indicate that the concrete is wet.

During periods of cooling weather, the concrete is likely to be several degrees warmer than the thin plastic sheeting. This will result in a migration of vapor moisture from the warmer concrete to the colder plastic. Care must be taken to see that the plastic sheeting is secured against being removed or torn by wind or carelessness. All edges and laps between sheets should be weighted sufficiently to guard against wind–caused air circulation under the sheeting.

A critical stage of the curing period occurs as concrete is passing from the plastic to the hardened state. During this period, which begins about an hour after placement, it is essential that concrete be protected from movement, drying, and sudden temperature changes. Concrete placed in slabs such as sidewalks or bridge decks during periods of windy or unusually warm dry weather may be vulnerable to cracking or surface checking. Slabs expose a large area per volume of concrete, which permits extremely rapid drying under the above conditions.

- Finishing

Finishing must keep pace with placement, followed by the application of curing compound, to prevent excessive drying of concrete surfaces. Once finishing is completed, curing compound can be immediately applied to hold moisture in the concrete.

- Concrete temperature

Although often considered separately, concrete temperature is an important consideration in the curing process. It must be measured and recorded at various times and locations to establish and document that specified temperatures and rates of temperature change are maintained.

Temperature measurements for the following materials are necessary:

- Mixing water
- Concrete at the time of placement
- Surface of previously placed concrete
- Concrete temperature during curing period

For measurement of concrete temperature during the curing period, location of temperature measurement points must be carefully worked out in advance. Widely varying temperatures are usually found in different parts of a concrete mass. The lowest temperatures will be found at the corners and near protruding reinforcement steel. The highest temperatures in the mass also should be observed as it may be the critical control item at the end of the curing period when temperature drop must be held to the specified 1°F per hour or less.

Temperature measurements in the concrete mass are made easily with thermometers inserted into wells or voids formed in the concrete at the time of placement. A good thermometer well can be formed easily by inserting a greased bolt through the form prior to concrete placement.

R. Hot Weather Concreting

The term “heat of hydration” is applied to the heat generated by the chemical reactions that occur in setting concrete between the water and cement. The heat causes the concrete first to expand and then to shrink as it cools. If there is a temperature gradient across the concrete or the concrete is otherwise restrained, cracking may occur. The two factors that are most likely to cause excessive generation of heat are very large pours and high cement contents.

More issues arise from concreting in hot weather than in cold weather. Of all the ingredients in concrete, water is the one that causes the most trouble in hot weather concreting. It evaporates faster, causing a rapid change in the concrete's volume, which leads to cracking. Therefore, the mix should be as dry as possible.

The newly–poured concrete must be finished as soon as is possible. When the surface is finished, steps should be taken immediately to cure the concrete. Taking precautions to lower the concrete temperature in hot weather is especially important for large pours. Cooling the mixing water down to 45°F can clip several degrees off the final temperature of the concrete. Adding ice to the mix instead of water will reduce the final temperature even more. Steps can be taken to keep stored water for concrete cool. Storage tanks, piping, and trucks should be insulated and their exteriors painted white. Also, aggregate stockpiles can be sprayed for cooling by evaporation.

The advantages gained by keeping materials cool will be rapidly lost if the concrete is over mixed or cast in hot forms. Over mixing can be averted by scheduling deliveries of truck–mixed concrete so that the trucks do not have to wait at the job site. Cement at a temperature of 150°F or higher should not be used because of the possibility of it forming into balls by rapid stiffening when it comes into contact with damp sand or small quantities of water. Refer to *Subsection 520.3.8.1 General* of the Standard Specifications for more information regarding temperature requirements for placing concrete.

For large footings, piers, etc., best engineering practices dictate pouring each structure all at once. If the concrete can't be kept cool and moist, then it must be kept warm. Differential temperatures should be minimized, and the outside surface temperature of the concrete should not differ from the ambient air temperature by more than 36°F. This means that the structure's surface should not be in contact with cold water if the internal concrete temperatures are high. This is to prevent thermal shrinking, which causes surface cracking.

The following formula provides a relatively close approximation of what the maximum internal temperatures of a large structure might be within the first three days of curing, when the heat of hydration is at its peak. Assume a design mix of 3,000 psi concrete, with ¾ in aggregate, and cement.

Given:

T_C = Temperature of concrete when placed (°F) = 82°F

P = Amount of cement in mix (lbs/yd^3) = 611 lbs/yd^3

Find:

T_{MAX} = Maximum temperature of hydrating cement (°F)

Solve:

$T_{MAX} = T_C + 23(P / 170)$

$T_{MAX} = 82 + 23(611 / 170)$

$T_{MAX} = 82 + 82.6647$

$T_{MAX} = 164.6647 \approx 165°F$

Therefore, the maximum temperature of the hydrating cement is 165°F.

Note: Concrete mix additives may cause this temperature to vary.

Wind, accompanied by low humidity and high temperatures, can cause shrinkage cracks to appear in the surface of bridge decks and pavement. In this event, it may be necessary to erect shade or wind breaks, or both, in extreme conditions, or even to postpone placing bridge deck or paving concrete until more suitable temperature conditions prevail, or at night, in order to avoid too rapid drying of the surface and severe shrinkage cracking. A deck should have the wet curing materials placed as soon as possible behind the final floating.

The following nomograph is used to estimate the evaporation rate of surface moisture from concrete. They are primarily used to determine if extra protective measures will be needed during and after a concrete deck placement.

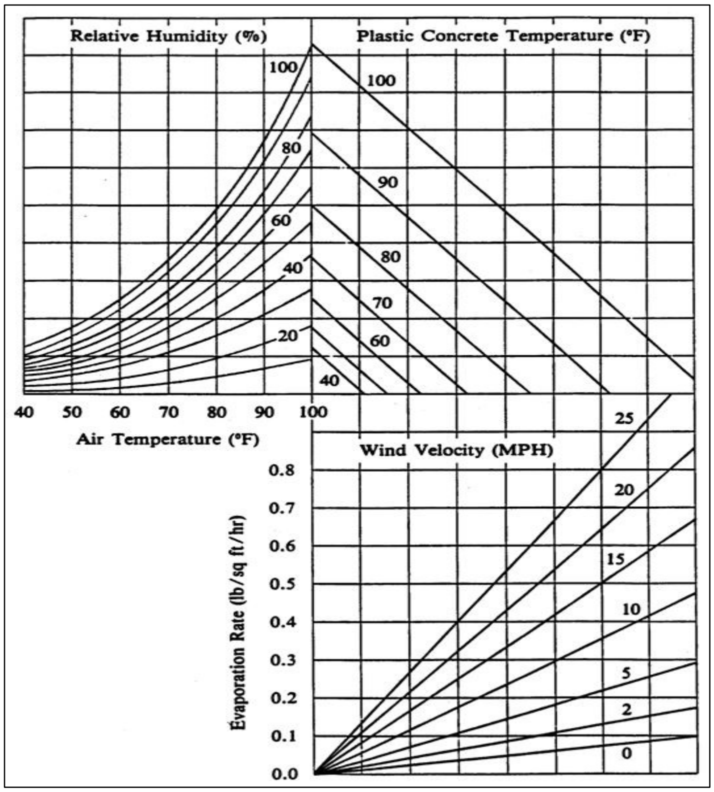


Figure 500 – 46: Nomograph for Calculating Evaporation from Free–Water Surfaces

S. Efflorescence

Efflorescence is a deposit, usually white in color that sometimes appears on the surfaces of masonry. When it does, it spoils the appearance. Often it is apparent just after the structure is completed – the time when the builder, architect, and owner are most concerned with the appearance of the new structure.

The efflorescence–producing salts found in masonry are usually sulfates of sodium, potassium, magnesium, calcium, and iron (ferrous), or carbonates of sodium, potassium, and calcium. Salts that are the chlorides of sodium, calcium, and potassium sometimes appear, but since these salts are highly soluble in water, the first rain will often wash them off the wall.

A combination of circumstances is necessary for efflorescence to appear. First, there must be soluble salts in the masonry. Second, moisture must pass through the concrete to act as a vehicle that will pick up the soluble salts and carry them to the surfaces. If either of these two conditions is eliminated, efflorescence will not occur.

In most cases, salts that cause efflorescence come from within the concrete itself. However, sometimes chemicals in the masonry react with chemicals in the atmosphere to form the undesired efflorescence.

Another source of salts is the soil in contact with at–grade placements and retaining walls. If these sections are not protected with a good moisture barrier, the salts may be carried up to the first few courses above ground.

Figure 500 – 47: Concrete Block Efflorescence

Identifying Efflorescence Issues: Since many factors can influence the formation of efflorescence, it is difficult to accurately predict the extent of efflorescence that may appear. However, as mentioned earlier, efflorescence will not occur if either the soluble salts are eliminated from the wall or water passage through the wall is prevented.

- Eliminating Salts: Methods for eliminating efflorescence–producing soluble salts in masonry walls are as follows:
  - Use only cementitious materials for mortar that meet the requirements of AASHTO M 85 Standard Specification for Portland Cement (Chemical and Physical).
  - Use only sand that meets the requirements of AASHTO T 71–08 Standard Method of Test for Effect of Organic Impurities in Fine Aggregate on Strength of Mortar and AASHTO M 6–3 Standard Specification for Fine Aggregate for Portland Cement Concrete. Unwashed sand shall not be used.
  - Use only masonry units of established reliability that pass the efflorescence tests in *ASTM* C 67 Standard Test Methods for Sampling and Testing Brick and Structural Clay Tile. Masonry units that have been observed to effloresce while stockpiled should not be used.
  - Use only clean mixing water that is free from harmful amounts of acids, alkalis, organic material, minerals, and salts. In some areas the drinking water may contain a sufficient quantity of dissolved minerals and salts to adversely affect the resulting mortar.
  - Use only insulating material that is free from harmful salts that may cause efflorescence for filling the cores of hollow masonry units.

- Verify that all mixers, mortar boxes, and mortarboards are free from contamination or corrosion. Masonry equipment should not be de–iced with any salt or antifreeze material.
- Verify that all tools are clean and free of rust, salts, and any harmful material. For example, do not use a shovel for any salt and reuse it for mortar sand without first thoroughly cleaning the shovel.

- Eliminating Moisture: Methods for eliminating moisture in and preventing its passage through masonry walls are as follows:
  - Flashings and copings should be properly installed to prohibit entry of water
  - Vapor barriers should be installed on exterior walls.
  - Barrier membranes or vapor–proof paint should be applied to interior surfaces.
  - Paint or other protective surface treatment should be applied to the outside surface of porous masonry units.
  - All mortar joints should be tooled with a V or concave–shaped jointer. This compacts the mortar at the exposed surface and helps improve the tight bond of mortar to the edges of masonry units. The use of weeping, raked, or untooled struck joints is not recommended.
  - Lawn sprinklers or other water sources should be properly located so that masonry walls are not subjected to unnecessary wetting.
  - Wide overhanging roofs should be used to protect masonry walls from rainfall if architecturally feasible.
- Removing Efflorescence: Most efflorescence can be removed by dry brushing. However, if this doesn't deliver satisfactory results, it may be necessary to wash the surface with a dilute solution of muriatic acid (5 to 10%). Before an acid treatment is used on any masonry wall, the acid should be tested on a small, inconspicuous portion to be certain that there is no adverse effect.

Before applying acid to the surface, the wall should be dampened with clear water. This will prevent the acid from being absorbed deeply into the wall where it may do substantial damage. After the acid treatment, the surface should be thoroughly flushed with clear water to remove all acid. Since an acid treatment may slightly change the appearance of masonry, the entire wall should be treated similarly to avoid discoloration or mottled effects.

A green stain may sometimes appear on buff or gray face brick or tile. This may be the result of vanadium or molybdenum compounds in the clay. Never treat such stains with acid, for the acid will react with these compounds to produce an insoluble brown stain that is extremely difficult to remove.

The proper method of removing these types of green stain is to wash the surface with a solution of one part sodium hydroxide crystals (lye) and ten parts water. The wall should be first dampened by spraying with clear water and then washed with the sodium hydroxide solution. A thorough flushing with clear water should follow.

Note: Rubber gloves and protective clothing should be worn by project personnel using an acid or sodium hydroxide solution to clean masonry walls.

Another source of staining is from granulated blast furnace slags used in concrete for improved permeability and strength properties. Staining with a bluish or greenish tinge may be noticed directly following form removal. This coloration is temporary and will likely disappear completely over time as the concrete cures. No cleaning or treatment for this type of staining should be attempted.

T. Finishing Unformed Surfaces

Use the following methods to finish the tops of footings, wall, piers, culvert floor slabs, and bridge decks.

- Tops of Footings, Walls, Piers, and Culvert Floor Slabs

These areas should be struck off and floated with a wood or magnesium float. The top surface of walls and piers should be struck off to a very accurate grade and floated to produce a relatively smooth, voidless surface. Culvert floor slabs should be struck off to the proper grade and floated with a wood float. There should be no low areas where water might collect.

Those areas of floor slabs and footings which are to be covered with concrete in a successive placement such as piers or culvert stems should not be floated to a smooth surface but should not be left in an extremely rough state as it is difficult to get complete coverage of such rough surfaces when applying curing compound.

For the rough surfaces at construction joints, there is too much surface area to cover with the normal rate of application of curing compound; consequently, these construction joints sometimes are not properly cured, resulting in weak concrete at the joint.

- Bridge Deck

  The following list has several possible causes of rough–riding bridge decks:

  - Improperly adjusted screed rails or screed guides
  - Poorly adjusted screeds and floats
  - Improperly set joint headers or expansion joints
  - Non–uniform consistency of the concrete, allowing the concrete to stiffen before finishing is accomplished

- o Inexperienced workers

The first straightedging of the bridge deck should be done while the concrete is still in a correctable condition. Any irregularities disclosed by the straightedging should be corrected immediately. Attention should be given to finishing the gutter lines on bridges, particularly on nearly flat grades, in order to preserve good longitudinal drainage. Do not over–work the finish. This is especially important on the deck copings. Over working causes surface spalling during freeze thaw cycles. Strike to grade, straightedge once or twice more, and cover all exposed concrete surfaces with wet burlap.

U. Field Testing

It is only by testing the concrete being used on the job that the most accurate determination can be made of the proper amount of each material to use in a mix. It is imperative that all project personnel engaged in inspection of concrete operations have a basic knowledge of the mechanics of mix proportioning.

The following factors should be considered when evaluating concrete mixes:

- Slump

  Subtracting 1 gal of water per yd^3 of concrete will generally decrease the slump by about 1 in.

- Air Entrainment

  Air entraining admixtures may usually be increased or decreased without changing other contents of the mix in order to obtain an allowable percent air value. However, these changes in air entrainment can adversely affect workability and strength and so should be controlled carefully. A change of aggregate or sand gradation may cause an air entrainment change. Therefore, changing the air entraining admixture dosage might help to correct the problem.

- Workability

  When a mix is unworkable and harsh, increasing the amount of sand in the mix by 50 lbs/yd^3 and decreasing the amount of stone by 50 lbs/yd^3 will result in a "creamier" consistency. If more sand is needed, the adjustment can be repeated again, but usually in 50 lbs/yd^3 increments. The workability of an over–sanded mix can be adjusted the same way in reverse. The yield and strength should be checked after any changes to the mix have been made.

- Water/Cement Ratio

  When adjusting concrete mixes as previously described, the water/cement ratio stated in the Specifications should not be exceeded. It is often more feasible to ensure that the water/cement ratio is not exceeded by knowing the maximum amount of water per cubic yard for the given concrete mix beforehand. Since the total delivered water is indicated on the plant's delivery slip, the Concrete Placing Inspector can quickly calculate how much additional water may be added at the work site.

- Yield

After the design mix has been prepared and approved, problems with yield can develop at the batching plant. Contamination of the aggregates by materials of different weights may result in changes in yield, since, in batching by weight, the volume of aggregate will change. For example, if lightweight organic material were to contaminate regular weight aggregates, a given weight of the aggregates would produce a greater volume than anticipated.

When the cement content is kept constant and yield increases, the cement factor is effectively lowered, and the result will be lowered strength. Conversely, if aggregates are contaminated with material of greater weight, the yield will be reduced and the strength increased. As an example, assume that a stockpile of structural lightweight aggregates accidentally becomes contaminated with regular weight aggregates. When the aggregates are batched, the weight will not supply nearly the volume anticipated and the yield will be reduced.

If yield is to be maintained, the moisture content of both the fine and course aggregates must be carefully monitored. If weights in the mix design are predicated on aggregates in one condition of saturation (SSD) and they are batched in some other condition (without appropriate adjustments), the result will be a change in the aggregate volume batched and therefore a change in yield as well. Failure to adjust the amount of water in the mix will also cause loss of slump and workability or lowered strength.

This matter is especially critical in lightweight aggregate concrete where absorption can run as high as 12%. Without careful monitoring, this can result in major variations in yield. Consider the following lightweight concrete mix design that is based on Saturated Surface Dry (SSD) aggregate weights:

Concrete Mix Design Based on SSD Aggregate Weights		
Amount	Units	Material
564	lbs	Cement
1,300	lbs	Fine Aggregate (Sand)
900	lbs	Lightweight Coarse Aggregate
29	gals	Water
6	%	Entrained Air
2–4	in	Slump

In the field, however, it turns out that the sand has a free moisture content of 5% and the lightweight coarse aggregate has a free moisture content of 10%. Applying compensating adjustment coefficients, the mix should be batched with 564 lbs of cement, 1,365 lbs (1,300 × 1.05) of sand, 990 lbs (900 × 1.10) of coarse aggregate, and 10.4 gals of water (155 lbs of free water equaling 18.6 gals already existent in the material).

If the mix is batched according to the weights given in the mix design (564 lbs, 1,300 lbs, and 900 lbs, respectively), the actual cement and aggregate weights would be 564 lbs of cement, 1,238 lbs (1300 / 1.05) of sand, and 818 lbs (900 / 1.10) of coarse aggregate.

This would occur because there would be 62 lbs of water in the sand and 82 lbs of water in the coarse aggregate, for a total of 144 lbs (17.3 gals) of extra water. Only about 11.07 gals of additional mix water would be needed to produce the required 2 to 4 in slump. In terms of yield, this would result in a batch amount about 5% less than specified in the design mix.

When preparing a mix design for regular weight concrete, the effect of air entrainment on yield should be considered. In regular weight concrete, each additional percent of air entrained causes a one lb/ft^3 reduction in concrete weight, while also causing an increase in yield.

When concrete delivery trucks are forced to wait long periods before they can discharge their cargo, there may be a reduction in yield due to loss of air and water. In this case, the Contractor must assume the responsibility for loss in yield, as well as the other degradations of quality that accompany long waits at the job site.

Other practices that can cause in a loss of yield are over–vibration, which drives air out, and “soupy” mixes that have increased water content, resulting in lower unit weights. Any practice that affects the water or air content of a mix, such as adding water at the job site or allowing concrete to dry out, will affect yield.

Certain types of concrete castings will also result in a loss of yield, even with excellent placement practices. For example, columns, high walls, mass concrete, and other structural applications involving considerable concrete pressure will result in a loss of yield, often as much as 10%. This loss is due to the high degree of density achieved in such work and the attendant loss of air content. If the loss of intentionally entrained air is not excessive, this will naturally produce superior concrete. However, the loss of yield should be kept in mind and measures to correct for these situations must be considered.

To minimize problems with concrete yield, Contract Administrators should verify the following items:

- The mix design is consistent with a yield of ±27 ft^3 of concrete per cubic foot of designed mix. The formula for this calculation is shown on the NHDOT concrete batch delivery slip (refer to previous examples).
- The Contractor has ordered a sufficient amount of concrete.
- The Contractor has scheduled deliveries that minimize delays when discharging concrete from delivery trucks.
- The concrete forms are constructed mortar–tight.
- The methods of transporting and placing equipment will avoid over–vibration, segregation, and other problems.

Obviously, it is of great importance to maintain a correct yield. All parties benefit when the yield parameters are kept in bounds and all stand to lose when they are allowed to vary. Responsibility for variations in yield can be traced to conditions or practices before and/or after delivery of the concrete. In almost every case, the change in yield will be accompanied by changes in important characteristics such as concrete strength and durability.

Yield of the concrete as delivered at the job site can be checked by the Inspector by regular measurement of unit weight. If this consistently varies more than 2%, an investigation should be made into such factors as air content, aggregate weight, or batch weights. When the unit weight varies considerably, it will often be found that the yield, as well as other concrete properties, also varies.

To check the actual volume of concrete being delivered to the job, divide the plastic unit weight of the concrete into the total number of pounds of all the mix ingredients.

V. Concrete Mix Design

To adjust a concrete mix design, it is best to be familiar with the initial design of concrete mixes. Data required to design a concrete mix would ordinarily come from field and laboratory testing.

The following tables may be used in lieu of some items of field data.

Volume of Coarse Aggregate per Unit Volume of Concrete				
Maximum Size of Aggregate (in)	b/b_o: Volume of Dry Rodded Coarse Aggregate Per Unit Volume of Concrete for Different Fineness Moduli (FM) of Sand			
Fineness Modulus	2.40	2.60	2.80	3.00
⅜	0.50	0.48	0.46	0.44
½	0.59	0.57	0.55	0.53
¾	0.66	0.64	0.62	0.60
1	0.71	0.69	0.67	0.65
1½	0.75	0.73	0.71	0.69

Water–Cement Limits in Different Classes of Concrete					
Concrete	Minimum Compressive Strength[1]	Minimum Amount of Cement per yd^3 of Plastic Concrete		Gallons of Water[2] Air Entrained Concrete	
Class	lbs/in^2	lbs	bags	gals per bag	Water/ Cement Ratio
AA[3]	4,000	658	7.0	4.29	0.380
AA	4,000	658	7.0	5.00	0.444
A	3,000	611	6.5	5.23	0.464
B	3,000	564	6.0	5.50	0.488
T[4]	3,000	620	6.6	6.30	0.559

[1] Minimum compressive strength expected at 28 days when cured in a moist room at a temperature between 65 and 75°F

[2] If more water is required than shown, the cement factor shall be increased to maintain the desired water–cement ratio, or the concrete mix shall be redesigned

[3] When high range water–reducing admixture is used

[4] Fly–ash is not allowed in Class T Concrete

The follow table shows a sample concrete mix design.

State of New Hampshire / Department of Transportation				
Materials and Research Bureau				
Sample Concrete Mix Design (1)				
Project:	Laconia		Lab No.:	391X96
Federal No.:	NHS–018–2 (104)			
State No.:	99999			
Report to:	☒ Contract Administrator	Ronald Tanner	☐ FHWA	☒ Lab
	☒ Other		☐ Construction	

Mix Type	☐ Non–Vibrated		☒ Vibrated		☐ Air Entrained		Slump	2 – 4 in
Class	AA ☒	A ☐	B ☐	C ☐	T ☐	P ☐	Max. Water/ Bag Cement	5.0 gals

Source of Fine Aggregate:	Coastal–Farmington			
Source of Coarse Aggregate:	Coastal–Raymond			
Type of Coarse Aggregate:	Gravel ☐	Rock ☒	Chemical Admix. Req. ☐	WRA or Retarder ☐
Size of Coarse Aggregate (in):	#4 – ¾ in			
Fine Aggregate: FM	2.65	Absorption	0.746%	
Sp. Gr. (Sat. Surf. Dry):	2.632	Solid Wt. (A)	168.0 lbs/ft^3	
Coarse Aggregate: Rodded Wgt. (C):	106.5 lbs/ft^3	Absorption:	0.438%	
Sp. Gr. (Sat. Surf. Dry):	2.687	Solid Wt. (B)	180.8 lbs/ft^3	

State of New Hampshire / Department of Transportation				
Materials and Research Bureau				
Sample Concrete Mix Design (2)				
Coarse Aggregate: Rodded Weight (C)		106.5 lbs/ft^3	Absorption	0.438%
Sp. Gr. (Sat. Surf. Dry)		2.687	Solid Wt. (B)	180.8 lbs/ft^3
Vol. Coarse Aggr. per Vol. of Conc. = b/b$_o$=		0.64 × 27 ft^3/yd^3 = 17.28 ft^3 (D)		
Wt. Coarse Aggr. per yd^3 of Conc. = (D)		17.28 × (C) 106.5 = 1,840.3 lbs (E)		
Solid Vol. of Coarse Aggr.		1,840.3 (E) / (B) 180.8 = 10.18 ft^3		
Solid Vol. of Cement =		7.0 bags × 0.478 = 3.35 ft^3		
Solid Vol. of Water =		35 gals / 7.49 = 4.67 ft^3		
Volume of Air =		7.0% × 0.01 x 27 = 1.89 ft^3		
Total Solid Volume except Sand =		20.09 ft^3 (F)		
Volume of Sand = 27 – (F) =		6.91 ft^3 (G)		
Weight of Sand = (G) × (A) =		1,160.9 lbs		
Ratio of Sand to Total Aggregates		% by wt.	Yield Adj. to Design Mix	☐
Batch Weights per yd^3 (lbs)				
Cement:	658.0			
Coarse Aggregate (40%):	736.0	#4 – ⅜ in		
Coarse Aggregate (60%):	1,104.0	⅜– ¾ in		
Coarse Aggregate:		¾” – 1½”		
Fine Aggregate:	1,161.0			
Total Water:	35.0	gals/ yd^3		
Wet Density:	146.3	lbs/ ft^3		
Respectfully	James J. Marol	Concrete Supervisor		
Respectfully	Alan D. Angelos	Chief of Materials Technician		
Date Reported:	July 31, 2015			

Note: Moisture content of Fine Aggregate and Coarse Aggregate should be determined and mix design adjusted prior to batching. Total weight of SSD Fine and Coarse Aggregates remains constant.

For Department mix designs, changes in the fine aggregates should not vary from the design FM by more than ±0.2. If a greater variation occurs, a redesign of the concrete mix might be required. Following the procedure used on the original concrete mix design, a redesign can be computed up to and including the ratio of sand to the total aggregate using the previous tables.

Field checks should be made to verify the percent of air entrained, the weight of concrete, and yield after any redesign has been performed. Adjustments should be made if necessary. See *Division 700* of this manual for concrete field sampling and testing procedures.

Factors affecting concrete compressive strength in clued the following:

- For concrete of a given slump and a given maximum size of aggregate, the addition of one bag of cement per yd^3, within the range of typically used cement factors, will increase the 28–day compressive strength by approximately 1,000 psi.
- For concrete having given proportions of dry ingredients, each increase of 1 in of slump will be accompanied by a reduction in 28–day compressive strength of about 200 psi.
- For properly proportioned mixtures of the same slump, doubling the maximum size of aggregate (say, from ¾ in to 1½ in) will increase 28–day compressive strength by about 400 psi.
- For properly proportioned mixtures, comparable as to cement factor and slump, air entrainment in usual amounts will affect strength approximately as follows:
  - In lean mixtures, no effect or slight increase in strength
  - In mixtures of intermediate richness, a reduction in strength of 2% to 3% for each percent of air
  - In rich mixtures, a reduction in strength of 4% to 5% for each percent of air

W. Bridge Deck Construction Checklist

The following checklist should serve as a "memory jogger" for all aspects of bridge deck construction.

Schedule QC/QA Pre–placement Meeting

- Review Process Control Plan (PCP) – Refer to *Subsection 520.4 Quality Control/Quality Assurance (QA/QC)* for more information

Falsework and Deck Forms

- Adequacy of Design:

  - Approved falsework plan on project and constructed falsework conforms to plan
  - Finishing equipment should be included with falsework plans
- Construction:
  - Overall length and width of formwork for the span, checked and verified to be correct
  - Deck form set to correct elevation with the blocking distances accommodating differences between actual and theoretical beam camber
  - All chamfer drip strips in place and securely nailed
  - Formwork free of construction debris
  - Deck forms mortar–tight
  - Bracing adequate at end dams and fascias
  - Utility structures not in interference with forms
  - Deck drains, scuppers, and bridge rail anchor bolt assemblies in proper locations
  - Safety check of equipment, guardrails, ladders, etc.
  - Reinforcement Steel, Steel Joints, Expansion Material
- Reinforcement Steel Bars:
  - Bars correctly spaced and securely tied
  - Bars positioned to provide proper concrete cover and form clearance
  - Bars clean and epoxy coating touched up
- End Dam Assemblies:
  - End dam assemblies checked for correctness in fabrication of dimensions, shapes, and straightness
  - End dam assemblies set to proper grade
  - Assemblies securely supported, securely attached, and shipping ties released
  - Expansion material cork, Korolath, bond break paper, and filler joints correct in dimension and securely placed
  - Screed Guides
- Adequacy of design:
  - Total load of all screeding equipment considered
  - Rail supports designed to permit their removal above the top of the slab

- Construction:
  - Screed rails set to correct elevation and alignment
  - "Dry Run" pass of screeding equipment over the screed rails performed, verifying that the rail support spacing is adequate to prevent deflection of the rail at intermediate points
  - "Dry Run" pass of screeding equipment over the screed rails performed, verifying that the proper elevation tie–in has been made at the span ends with the steel joints or the backwall of the abutment and deck thickness
  - "Dry Run" is also performed to verify the proper concrete cover depth over the top mat of reinforcing steel
  - Rails checked by eye for symmetry of cambered structure

Equipment Planning

- Placement and Consolidation:
  - Concrete placement equipment, crane, chutes, pumps, etc. tested for operational condition
  - Platforms in place to blanket the ground beneath the crane bucket during discharge of concrete from the truck or mixer
  - Adequate number of properly functioning vibrators present with one or more in reserve
  - Hand tools present
  - Sample testing equipment available and adequate for project requirements
  - Inclement weather station and functional curing box
- Finishing:
  - Screed checked for operational condition, particularly the straightness and cleanliness of the strike and screed bar edges
  - Hand finishing tools and equipment present and free of any hard paste
  - Work bridge is present, capable of facilitating access by finishers to any point on the deck
  - Lighting equipment, including power source, present and checked when the finishing operations are likely to occur after dark
- Curing and Protection:
  - Equipment present for application of curing materials
  - Adequate covering present to protect freshly–poured concrete surfaces from damage by unexpected rainfall

- Adequate heating equipment present to maintain specified temperature ranges during cold weather concreting

- Personnel Planning
  - Competent bridge superintendent to be present throughout the concrete pouring operations
  - Adequate number of finishers available as scheduled
  - Adequate number of vibrator operators available as scheduled
  - Adequate number of laborers available as scheduled to place the concrete, operate the screed, and "serve" the finishers
  - All project personnel are fully informed of their duties
  - Adequate state personnel available as scheduled for on–site or plant testing

Materials Planning

- Concrete:
  - Concrete producer provided with concrete consistency specifications
  - Concrete producer provides assurance that the delivery process will be adequate to maintain continuous and uninterrupted placement
  - Communication established between the job site and the batching plant
  - Access road to the bridge construction site is in satisfactory condition to cause no delay
- Admixtures:
  - Admixture manufacturer appears on the Qualified Products List
  - Adequate amount of admixture on hand to accommodate pour
- Curing Materials:
  - Adequate quantity of concrete curing material present
  - Temperature ports available for sampling concrete temperatures
- Concrete Placement:
  - Preliminary handwork completed satisfactorily before machine progresses
  - Finish machine and operator performance OK
  - Vibration of mix properly performed
  - Placement of concrete not too fast for machine.
  - Samples and test made, with any adjustments made accordingly
  - Proper clearances of reinforced steel verified as finisher progresses

- Work bridge and finishing at sufficient interval behind finish machine
- Curing compound and wet curing materials applied at proper time

Sample Concrete Pre–Placement Checklist					
Project	Laconia NHS–018–2 (104)		Location	South Abutment & Wing Footings	
Computed Quantity		74 yd^3	Date Placed	September 1, 2015	
Date	Initials	Description			Comments
09/01/15	MB	1. Vertical and horizontal alignment			
09/01/15	MB	2. Exposed face of forms (dented or damaged, oiled, all holes plugged)			
09/01/15	MB	3. Form dimensions (tightened mortar tight & chamfer strips installed if necessary)			
09/01/15	MB	4. Tie screws & bracing (all dimensions)			
		5. Oil contraction joint			N/A for this pour
		6. Waterstop location & proper installation			N/A for this pour
		7. Weeper pipe location & installation			N/A for this pour
09/01/15	MB	8. Reinforcing steel & proper laps (in place) line and grade			
09/01/15	MB	9. Clearance of re–steel and ties			
		10. Bridge seat location, grades, and proper location of re–steel for anchor bolt			N/A for this pour
09/01/15	MB	11. Grades for top of concrete			
09/01/15	MB	12. Clean bottom of form			
		13. Tremie location			N/A for this pour
09/02/15	MB	14. Curing compound and protective			Two available
09/02/15	MB	15. Working vibrators (rubber coated)			
09/02/15	MB	16. Thermometer ports			
09/02/15	MB	17. Key ways prepared			
		18. ______________________________			

520.4 – QUALITY CONTROL / QUALITY ASSURANCE (QC/QA)

Quality is defined by our customers, the traveling public. It can be defined as the degree of excellence of a product or service, the degree to which a product or service satisfies the needs of the customer, or the degree to which a product or service conforms to a given requirement or expectation. The level of conformance to the customers' requirements is the measure of quality.

NHDOT contracts that include "QC/QA" Specifications also include Quality Based Price Adjustment Clauses. These Specifications provide financial disincentives for work with quality that is marginally less than desired and incentives for work that is above and beyond the Department's expectations. Here, the responsibility for providing the control of material quality is left in the hands of the project's Contractors, Subcontractors, suppliers, fabricators and manufacturers.

Quality Control (or Process Control) is a comprehensive, systematic and continuous approach to producing and placing consistent materials that will perform within Specification limits. Quality Assurance is a system for measuring the material quality provided based upon a statistical analysis of random testing of the finished product performed by the Department.

As many aspects of a product cannot actually be quantified so as to be measured and evaluated it is still necessary for NHDOT personnel to be actively engaged in overseeing the work. While it is true that the Contractor is solely responsible for some quality control processes, it is essential for NHDOT personnel to actively participate in the planning and execution of the construction to provide Quality Assurance and acceptance. Ensuring that quality is built into the project while it is happening is often the only means of obtaining the desired results.

Under QC/QA Concrete Specifications certain quality aspects (i.e., percentage of entrained air, water/cement ratio, concrete cover, strength, and permeability) are identified as items to be controlled by the Contractor and/or supplier in accordance with a quality control plan that they submit to the Department for approval. It is the NHDOT's responsibility to ensure that the plan contains all of the measures that will result in a final product meeting the criteria set forth in the Specifications.

Contractors shall submit a project–specific Process Control Plan (PCP) at least three weeks in advance for review by the Department. This plan should outline their organizational structure of personnel, which should include the following parties:

- QC Plan Administrator
- Production Facility managers and staff
- Field Technicians
- Field Inspectors
- Laboratory personnel

- Consultant testing agencies

The PCP shall reference applicable specifications, provisions, and authorities, including the following:

- Standard and Specifications
- Supplemental Specifications
- Special Provisions
- AASHTO
- ASTM
- ACI
- NETTCP
- Project drawings

The PCP shall address material control issues, including the following:

- Materials sources
- Material properties
- Mix designs
- Storage
- Stockpiling

The PCP shall address sampling and testing issues, including the following:

- Lot and sublot sizes
- Sampling plan
- Sample identification
- Reporting
- Action limits

The PCP shall address facility management issues, including the following:

- Schedules
- Equipment
- Activities
- Control charts

- Non–conforming materials
- Corrective action
- Production inspection

The PCP shall address field management issues, including the following:

- Schedules
- Equipment
- Control strip procedures
- Activities
- Material transport
- Delivery and placement
- Finishing and curing
- Corrective action for non–conforming materials
- Field inspection
- Rejection criteria

The PCP should also address any other plans or contingencies, including working with Subcontractors and suppliers.

The Process Control Plan (PCP) will be the document that all parties will refer to before, during and after the work is performed.

A successful Quality Control system does not rely just on the plan, but also continuous periodic formal inspection at all levels. It is imperative that those performing the work have a genuine knowledge and understanding of their responsibilities and how their actions affect the desired end product.

QC personnel are expected to test their products on a continuous basis to ensure quality requirements are being met. Department personnel, as stated earlier, will randomly sample and test the end product for Quality Assurance. The results of these QA tests will be used not just for pass/fail, but to make assertions about the Quality Level of the total material quantity. This Quality Level will then be used by Department personnel to make Pay Adjustments, positive or negative, to the bid prices.

The following analysis is used to determine Pay Factors for each characteristic being measured and is also outlined in the Specifications:

1. Determine the average (X) of the test results.

Determine the standard deviations of the test results.

Compute the Upper Quality Index:

$Q_U = (USL – X) / s$,

where

USL = Upper Specification Limit

Compute the Lower Quality Index:

$Q_L = (X – LSL) / s$,

where

LSL = Lower Specification Limit

Determine the Percent Within Upper Limit (P_U) from *Table 106–1 Quality Level Analysis by the Standard Deviation Method* of the Standard Specifications.

Determine the Percent Within Lower Limit (P_L) from *Table 106–1 Quality Level Analysis by the Standard Deviation Method* of the Standard Specifications

Compute the Quality Level (total percent within specification limits):

$QL = (P_U + P_L) – 100$

Refer to *Table 106–2 Pay Factors* of the Standard Specifications to determine the Pay Factor (PF) associated with this Quality Level.

Calculate the corresponding Pay Adjustment as follows:

Given:

$PA = PF \times Q \times UP$

where:

PA = Pay Adjustment

PF = Pay Factor

Q = Quantity in the lot

UP = Unit Bid Price

When the calculated Pay Factor is less than 0.75, the Department will require that no pay adjustments be paid and that the Contractor submit an engineering analysis showing either that the work will perform satisfactorily if left in place or a detailed plan for corrective action. The Department will review the analysis and, if the work is found to present little detriment to the serviceability of the structure, arrive at a negotiated settlement (credit) to be received from the Contractor.

QC/QA Pre–Placement Meeting Agenda Items (1)	
Project:	
Date:	
Location:	
Date of Placement	
Rain Date	
Start Time	
Total Quantity, # of Sublots	
Specification Date	
Delivery Rate, Route	
Method of Placement	
Expected Air Loss Pumping	
Direction	
Truck Access, Travel Time	
Washout Pit Location(s)	
Traffic Control	
Backup System	
Weather Station	
Lighting Placement	

QC/QA Pre–Placement Meeting Agenda Items (2)	
Bad Weather Protection	
Curing System	
Sidewalks & Brush Curbs	
QC Personnel (Site, Plant)	
QA Personnel (Air, Cyls, µ)	
Air Meter Calibration	
Early Break Cylinders	
Testing Location, Access	
Info Air Test 1st Load	
Split Samples (Schedule)	
Request Radar 3 Day Notice	
Add'l µ, Pyrex, Curing Box	
Distribution List & Fax Nos.	
Other Issues:	

SECTION 528 – PRESTRESSED CONCRETE MEMBERS

528.1 – GENERAL

The Materials and Research Bureau is the central point of contact for precast/prestressed concrete issues that occur between the time of project award and the in–place installation of the members. Per an inter–department memo dated January 8, 2003 from bureau administrators to bureau personnel, the various parties' responsibilities are as follows:

A. Materials and Research Bureau

- Review and approve the concrete mix design. A copy of the approved mix design should be forwarded to the Bridge Design Project Engineer (PE) and District Construction Engineer (DCE).
- Determine whether a test section is required and indicate if it is needed on the mix design approval.
- Generate the Plant inspection assignment letter to the Consultant Inspector. Copies should be provided to the Construction Bureau and the Precast Plant.
- Forward one copy of the approved shop drawings to the Concrete Plant Inspector.
- Notify the Contract Administrator if problems during production could affect the delivery schedule.
- Request a repair procedure from the Precaster as necessary, and determine the level of inspection required for repairs. All structural repairs shall also be reviewed by the PE & DCE. Materials and Research will provide the approval to proceed with the repair and copy Bridge Design & Construction.

Conduct independent assurance inspection of Consultant Inspectors.

Figure 500 – 48: Prestressed Concrete Members before Installation

B. Construction Bureau

- Highlight the precast submission timeline requirements at the preconstruction meeting.
- Six copies of shop drawings shall be submitted to Construction (the detensioning sequence shall also be submitted on a separate sheet of paper – for informational purposes and not for approval). Forward all copies of shop drawings to Bridge Design for approval. Bridge Design returns three copies to Construction (one for the Contractor, one for the Contract Administrator, and one for the Precaster), and two copies to Materials and Research Bureau (one for Materials and Research and one for the Inspector).
- Forward the Contractor's mix design to Materials and Research.
- Coordinate Pre–Placement Meeting 45 days prior to casting.
- Report all damage that occurs during erection directly to Materials and Research. Damage shall be thoroughly documented using Department forms, and pictures shall be taken of the damaged areas.
- Include a copy of the Consultant Inspector Report in the project files.

C. Bridge Design Bureau

- Review and approve shop drawings; forward 5 approved stamped copies (3 to Construction and 2 to Materials and Research).
- At the request of Materials and Research, Bridge Design will review damaged members to determine whether the structural integrity has been compromised and recommend a repair procedure if appropriate. Recommendations should be forwarded to Materials and Research with a copy to Construction.
- Review Precaster repair procedures.

D. Consultant Concrete Plant Inspector

- Coordinate with the precast plant to determine casting schedule, and communicate this information to the Bureaus of Construction and Materials and Research.
- Inspection: Members shall be inspected by the Concrete Plant Inspector during fabrication, at loading for shipment, upon arrival at the site, and during repairs required due to damage observed at any of these stages of inspection.
- Reject unsatisfactory materials or workmanship during fabrication and prior to transport, in accordance with the Specifications.

- Report all damage during fabrication and transportation directly to Materials and Research.
- Notification process:

Members damaged during fabrication

  - Report directly to Materials and Research.
  - Use the PCI NE Bridge Member Repair Guide as a reference.
  - Immediately notify Materials and Research of damage, which shall be thoroughly documented using Department–supplied forms. Pictures shall be taken of the damaged areas. This information shall be provided to Bridge Design and Materials and Research Bureaus as requested.

Members damaged during transport

  - Concrete Plant Inspector reports to Materials and Research and informs Contract Administrator verbally.
  - Damage shall be thoroughly documented using Department supplied forms. Pictures shall be taken of the damaged areas.
  - Members shall be repaired only as approved, with all work performed by qualified personnel. Repairs shall take place in the presence of a Department Inspector.

At the completion of the inspection assignment, the Concrete Plant Inspector shall complete and submit two copies of the final report that contains all records relevant to the assignment, such as daily logs, material reports, etc. The report copies shall be forwarded to the Bureau of Materials and Research, who will forward one copy to the Bureau of Construction.

E. Precasters

- Submit the shop drawings, concrete mix design, detensioning sequence and any necessary repair methods, as required by the Specifications or as directed.
- Notify Materials and Research, 14 days prior, of intent to cast so plant inspection can be arranged.
- Notify Materials and Research two days prior to the actual start of casting.

SECTION 536 – EPOXY COATING

536.1 – GENERAL

This item consists of furnishing and placing epoxy resin protective coating systems on concrete surfaces. Some of these same materials are used as bonding agents for bonding new concrete to new or old concrete and for bonding new concrete to steel.

The following are typical epoxy coating applications used in New Hampshire:

- Waterproofing
- Concrete patching and repairing
- Resurfacing worn and damaged areas
- Bonding broken pieces of concrete back in position
- Bonding extruded curbs
- Bonding concrete placed in blockouts such as those used with modular expansion dams

Other applications for epoxy coatings include the following:

- Leveling bridge decks and sidewalks
- Leveling bridge seats for load distribution
- Bonding traffic buttons
- Bonding concrete to steel beams, dowels, and studs
- Making road surfaces skid resistant

The following are reasons for using epoxy coatings:

- Solvent resistant
- Acid resistant.
- Alkali resistant
- Salt resistant
- Waterproof
- Abrasion resistant

- Forms high–strength bonds with most materials
- Heat and cold resistant
- Cures rapidly (opening the area for traffic)

Note: These materials are used both for protective coatings and as bonding agents. Generally when used as a protective coating, epoxy is a pay item; when used as a bonding agent, it is subsidiary to Item 520. Check the plans and special provisions for any variations in this policy.

536.3 – CONSTRUCTION OPERATIONS

A strong reliable epoxy bond can only be achieved on a clean, dry concrete surface. As new concrete initially contains a considerable amount of water, some of which is surplus, the concrete must be permitted to age before any epoxy is applied. Epoxy should not be applied until most of the shrinkage has taken place. Epoxy mixes can be modified for use with fresh concrete, rapid curing concrete, and other concrete mixes, but the manufacturer's recommendations should be strictly adhered to in these cases.

A. Preparing New Concrete for Epoxy Coating

The surface of new concrete is always weak, no matter how strong the remainder of the concrete mass may be. This is a result of finishing the concrete, as vibration and troweling will bring the lighter components to the surface of the concrete. These lighter components, commonly called laitance, leave a smooth surface. This laitance is perhaps ½ in thick and is very weak in almost every respect. This laitance must be removed, as its presence limits the performance of anything applied to its surface.

Another surface condition results from the use of compounds applied to wet concrete surfaces in the form of a membrane to retard water evaporation while concrete is curing. These curing compounds are often fatty oils or resinous materials of the type that will act as a parting agent for any subsequent adhesive or coating. Most curing compounds are more or less invisible unless a very careful examination is made.

Even when traffic and weather have removed most of the curing compound, it is possible that some may be present on the more sheltered areas. Such curing compounds must also be removed. Sand blasting may be required to remove these compounds and prepare the surface. Typically, curing compounds should not be used anytime a subsequent seal coat is planned to be applied.

B. Preparing Old Concrete for Epoxy Coating

Old concrete may be frequently found to have weak, deteriorated surfaces. Even when the aggregate is exposed, its surface is often polished, which is an undesirable condition. The deterioration of the concrete is often accelerated by the use of de–icing salts during the winter. Removal of this weakened upper layer of concrete is necessary prior to the application of any epoxy coating.

Frequently, old concrete may be stained by petroleum–based agents which must be removed prior to the application of any epoxy coating. Grease cutting detergents or, in the case of stubborn materials, solvents such as Tolulene, an aromatic hydrocarbon, or High Flash Naphtha may be used to remove the foreign materials.

C. Application of Epoxy

Follow the details found in *Section 536 Epoxy Coating for Concrete* of the Standard Specifications and the manufacturer's instructions when applying epoxy as a waterproofing sealer. If the Contractor proposes to use a material other than what is exactly specified, check with the Construction Bureau office, Materials and Research Bureau, or the District Construction Engineer.

SECTION 538 – BARRIER MEMBRANE

538.1 – GENERAL

Barrier membranes are extremely important to the life expectancy of a structure and should be very carefully inspected. A properly applied barrier membrane will prevent deterioration of concrete from salt and water intrusion. The barrier membrane's manufacturer shall assign a manufacturer's representative to be present at all times during barrier membrane installation operations, including the application of primer and tack coat, as well as during placement of hot bituminous pavement overlay.

A Department–hired Consultant Inspector trained in inspection may be utilized to assist and provide technical expertise to the Contract Administrator for barrier membrane systems installed on bridge decks. Inspectors should be contacted early enough in the process to allow for coverage during all aspects of membrane work, form beginning to end, starting with surface preparation of the bridge deck and ending with paving.

Consultants should be contacted in the order of their contracts. This information may be obtained from the District Construction Engineer or the Materials and Research Bureau. It is important that the authority of the Inspector be defined early in the process. Inspectors are hired to work for the Department, specifically for the Contract Administrator.

Figure 500 – 49: Barrier Membrane Installation

The barrier membrane Specifications outline certain quality control tests to be conducted by the Contractor and/or applicator during performance of the work. On newer projects, additional quality control procedures are documented in the membrane manufacturer's Field Process Quality Control Plan. This plan must be approved by the Materials and Research Bureau prior to the commencement of any membrane preparation or placement. Unless otherwise noted in the Specifications or otherwise ordered by the Engineer, the Department's consultant Inspectors should be conducting random quality assurance tests rather than more frequent quality control tests for the Contractor.

All necessary membrane testing materials as noted in the Specifications should be on site, such as the moisture meter, rubberized surface comparison chips, and the adhesion tester. Refer to *Technical Guideline No. 310.2R–2013 – Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair*, published by the International Concrete Repair Institute (ICRI) more information regarding concrete surface preparation.

538.2 – MATERIALS

Materials needed for standard sheet membrane consist of an adhesive primer, preformed waterproofing membrane sheet, and mastic. Liquid spray applied membrane consists of a primer, a two–part barrier membrane system (usually applied in two layers with a different colored membrane for each layer), and a final primer to bond with the asphalt. Torch applied membrane consists of a primer and an "asphaltic" sheeting membrane that should be placed by automation.

Heat generated by automated or manual torches and the pavement is used to create the bond between the torch applied membrane and the primer and pavement, respectively.

Hot rubber is often used to seal the curb lines for liquid and torch applied membrane. All components should be compatible as recommended by the manufacturer, and must be on the Qualified Products List. If the Contractor wishes to use a product not on the Qualified Products List, a request and sample shall be submitted to the Materials and Research Bureau for approval. It is imperative that the correct type of membrane is specified for use on vertical and horizontal surfaces.

538.3 – CONSTRUCTION OPERATIONS

Before performing any work, the Contractor should be advised to protect all parts of the structure near the work area from being smeared with primer or mastic (i.e., granite curbing, catwalks and sidewalks, bridge railing, etc.). Primer and mastic smears, hardened liquid membrane, and charring from the torches from torch applied membrane are nearly impossible to clean satisfactorily and the Contract Administrator should insist that the waterproofing be applied in a workmanlike manner.

If the Contractor fails to comply with this request, additional measures must be taken such as covering exposed areas before proceeding with the work. The entire surface to be covered with barrier membrane should be cleaned of all foreign materials such as oil or grease. Any sharp protrusions shall be removed to create a reasonably smooth surface.

Most new deck membranes require that the deck be shot–blast cleaned to remove any laitance. The resulting surface should be checked against the previously referenced *Technical Guideline No. 310.2R–2013* of the International Concrete Repair Institute and concrete surface profile chips, which are rubberized replicas of typical concrete surface conditions. If the membrane is being applied on structures that were previously waterproofed, all existing waterproofing shall be removed from the surface before applying any new waterproofing.

No vehicles other than paving equipment should be allowed on the membrane–covered surface prior to pavement overlay. Overlay equipment wheels and tires should be cleaned and free from stones and other material that could penetrate the membrane. A light dusting of portland cement or talc may be used on the membrane in the wheel paths to prevent sticking during hot weather paving operations. If paving on top of the liquid applied membrane, a soapy water solution should be sprayed on the paver wheels to prevent delaminating the paving primer from the cured membrane.

The membrane is to be covered within three days as required in *Subsection 538.3.3.5 Application of Hot Bituminous Overlay* of the Standard Specifications. Since the application of a barrier membrane not only forms a water seal, but also an air seal, it may accumulate air bubbles. For standard sheet membrane, this trapped air can be released by taking a knife and perforating the bubble at its base just prior to paving. The heat from the hot bituminous pavement will melt and reseal the puncture when rolled.

Any air bubbles in spray–applied membrane must be released before applying the paving primer by cutting a slit and then sealing with a coating of the same liquid membrane mixture, usually applied with a print brush. Similarly, holes and "holidays" are fixed the same way.

SECTION 544 – REINFORCING STEEL

544.2 – MATERIALS

The manufacturer's delivery slips should be checked against the Department's reinforcement bar list, and any discrepancies should be investigated and corrected, if necessary. Payment for this item will be according to the contract "F" item total if applicable. If the item is not a final pay (F) item, then it will be based on the corrected delivery slip totals.

Figure 500 – 50: Reinforcing Steel Installation

544.3 – CONSTRUCTION OPERATIONS

The reinforcing steel should be stored on supports to prevent excessive sagging of the bundles and so that the steel does not come in contact with the ground. The Contractor should be encouraged to store the steel with the shipping tags attached. When steel is stored for periods longer than two months, it should be protected from the weather with a covering. If plastic sheeting is the covering used, it should be draped over the bundles while still allowing air to circulate around the steel to minimize condensation under the plastic. Any covering placed over epoxy–coated reinforcing steel should be opaque to protect the epoxy from exposure to the sun.

A. Reinforcing Steel Markings

When the bars arrive on site, check the identification marks stamped into the steel to be sure they correspond with specification and plan requirements.

The following *ASTM* Specifications for reinforcing steel bars in general use on NHDOT projects are as follows:

- ASTM A 615 Standard Specification for Deformed and Plain Carbon–Steel Bars for Concrete Reinforcement
- ASTM A 706–15 Standard Specification for Deformed and Plain Low–Alloy Steel Bars for Concrete Reinforcement
- ASTM A 996 Standard Specification for Rail–Steel and Axle–Steel Deformed Bars for Concrete Reinforcement

Billet–steel, low–alloy steel, rail–steel, axle–steel reinforcing bars feature identification marks rolled into the surface of one side of the bar to denote the producer's mill designation, bar size, type of steel, and minimum yield designation.

- Grade 60 bars show these marks in the following order:
- Producing Mill (usually a letter)
- Bar Size Number (#3 through #18)
- Type of Steel:
- **S** for Billet Steel – Supplementary Requirements (S–1) of ASTM *A 615*
- **N** for Billet Steel – ASTM *A 615*
- **W** for Low–alloy Steel – ASTM *A 706–14*
- **R** for Rail Steel – meeting Bend Test requirements of ASTM *A 996* Grade 60 [per ACI 318–83]
-  for Rail – ASTM *A 996*
- **A** for Axle–Steel – ASTM *A 996*
- Minimum Yield Designation:

  Minimum yield designation is used for Grade 60 bars only and may be either–one single longitudinal line (grade line) or the number 60 (grade mark). A grade line is smaller and is located between the two main ribs, which are on opposite sides of all bars made in the United States. A grade line must be continued through at least 5 deformation spaces, and it may be placed on the side of the bar opposite the bar marks. A grade mark is the 4th mark on the bar. Grade 40 and 50 bars are required to have only the first three identification marks, with no minimum yield designation.

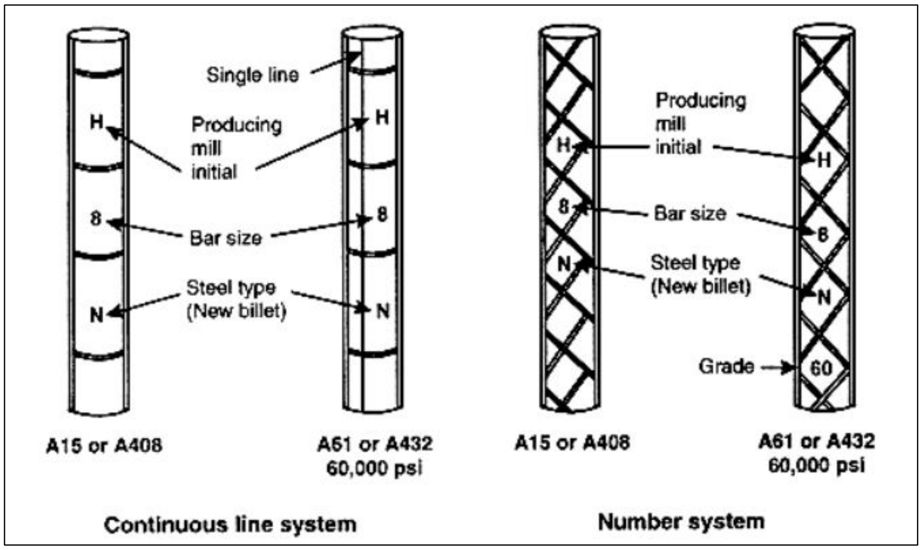


Figure 500 – 51: Reinforcing Steel Bar Markings

Bar identification marks may also be oriented to read horizontally (at 90° to those illustrated above). Grade mark numbers may be placed within separate consecutive deformation spaces to read vertically or horizontally. For more information on marking requirements for reinforcing steel, the Concrete Reinforcing Steel Institute (CRSI) website may be found at the following URL:

http://www.crsi.org/index.cfm/steel/identification

B. Reinforcing Steel Placement and Support

Reinforcing steel should be secured in its final position in the forms before the concrete placement begins. No reinforcement, such as dowels, should be permitted to be shoved into newly–poured, plastic concrete, since its bond strength will be jeopardized.

Checking spacing between reinforcing bars in the forms can be a tedious undertaking, especially with closely spaced bars. Generally, it is acceptable to lay off a space of about 3 ft and count the total number of bars in that space. For instance, if bars are called for at 9 in on center for 4 ft 6 in, verify that there are six fairly regular spaces instead of measuring every 9 in. Minor variations in individual spacings are seldom of significance so long as the total amount of steel is correct.

Figure 500 – 52: Reinforcing Steel Installation for Bridge Construction

Welding reinforcing bars to shear connectors, scuppers, end–dam assemblies, and rail bolt sets is prohibited. The Bridge Design Engineers and District Construction Engineer should be consulted if any such requests are made by the Contractor for legitimate reasons. Welding is not permitted on any main reinforcing steel. Welding, when authorized, should conform to *Section 550 Structural Steel* of the Standard Specifications.

The Specifications call for Grade 60 bars, but Grade 40 is sometimes specified at certain locations. Check the project plans and Special Provisions carefully for usage of grade bars.

All chair and bar supports shall be estimated and furnished for bridge decks to allow for the minimum concrete cover of the reinforcing bars called for on the plans, and to ensure the spacing between supports does not exceed 4 ft.

For transverse bars, two continuous lines of bar supports per bay shall be provided for spans (centerline of beam to centerline of beam) up to 9 ft, with lines positioned at the 1/4 and 3/4 points. For spans greater than 9 ft, use three continuous lines of bar supports per bay with lines positioned at the approximate 1/6, 1/2 and 5/6 points.

Note: Bar supports are not intended to and shall not be used to support hoses for concrete pumps, runways for concrete buggies, or similar loads.

The nominal height of the bar support shall be taken as the distance from the bottom of the leg or runner wire to the bottom of the reinforcement. Variations of ⅛ in from the stated nominal height shall be permitted.

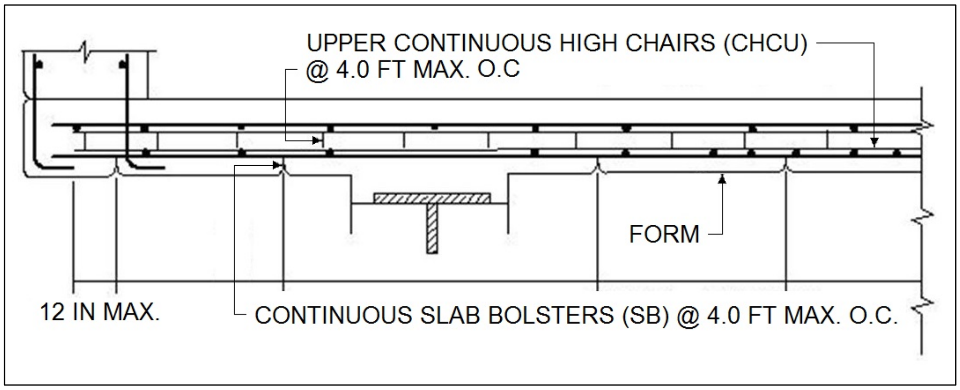
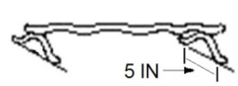
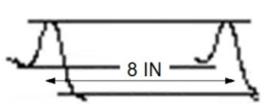


Figure 500 – 53: Slab Bolsters and Continuous High Chairs

The most widely used bar supports are standardized factory–made wire supports. Wire supports may be made from cold–drawn carbon steel or stainless steel wire. The lower portions may be provided with special rust protection by a plastic covering or by being made of stainless steel (bottom portion only).

The bottom reinforcement shall be supported on Type SB plastic protected or stainless steel protected supports. The end of the bottom supporting wire shall be overlapped to lock the last legs of the adjoining units.

All chair and bar supports used for the installation of epoxy–coated reinforcing bars shall meet the Standard Specification requirements found in *Subsection 544.2.4 Epoxy Coated Reinforcing Steel.*

Steel Reinforcing Bar Supports			
Symbol	Bar Support Illustration	Type of Support	Standard Sizes
SB		Slab Bolster	¾ in, 1 in, 1½ in, and 2 in height in 5 ft and 10 ft lengths
CHCU		Continuous High Chair Upper	2 in to 15 in height in ¼ in increments

Note: Top wire on continuous supports, not otherwise designated as corrugated, may be straight or corrugated at the Manufacturer's option

Minimum Wire Sizes of Bar Supports						
		Minimum Wire Sizes				
		Carbon Steel			Stainless Steel	
Symbol	Nominal Height	Top	Legs	Runner	Legs	Geometry
SB	All	W5 Corrug.	W2.9	—	W2	Legs spaced @ 5 in O.C. and vertical corrugations spaced @ 1 in O.C.
CHCU	2 in – 5 in inclusive	W5.5	W5	W5	—	Legs @ 20° or less with vertical; All legs 8 in Max. O.C., with legs within 4 in of end of chair, and spread between legs not less than 50% of nominal height
	> 5 in – 9 in inclusive	W5.5	W5.5	W5	—	
	> 9 in – 15 in inclusive	W5.5	W8	W5	—	
Reference: AASHTO M 32M Standard Method of Test for Steel Wire, Plain, for Concrete Reinforcement						

Important: Workers are prohibited from walking on finished steel mats and encouraged to use walkways constructed on dunnage.

The correct clearance spacing of the reinforcing steel from the tension face of the concrete member is particularly important. Another important clearance check is the measurement of the deck reinforcing steel mat from the concrete surface. After setting the screed on the concrete finishing machine over several blocking points to final deck grade, run the machine over the entire mat and check the top mat clearance.

The epoxy coating on reinforcing steel is used to protect the steel from corrosion due to chemical reactions. This coating is to be abrasion resistant and flexible. The epoxy coating is applied by spraying the epoxy powder onto the bar and then heating the bar to cure the epoxy. Upon delivery, the coating should be carefully checked for any damage caused during transportation.

Since the epoxy coating is flammable, bars should not be cut by burning. After cutting, the bars should be touched up with a repair kit supplied by the manufacturer. Use the same repair kit to touch up any subsequent damage to the coating. Repair kits generally require mixing up of a two–part epoxy then brushing the epoxy onto the exposed steel. Before application, the damaged area should be cleaned of all rust and contaminants.

Contractors may favor a spray–applied epoxy, but this "spray paint" type epoxy is generally not approved by the manufacturer, and as such, should not be used as a substitute to the manufacturer–supplied repair kit. When reinforcing steel is epoxy–coated, plastic–coated tie wire and plastic–coated bolsters are required. The epoxy coating is slippery. Therefore, bars should be tied tight to prevent their movement. When compacting the concrete, only rubber or non–metallic vibrators should be used to consolidate the mix, as a metallic vibrator may cause damage to the epoxy–coating bars within the concrete.

For sampling of reinforcing steel, refer to *Division 700 – Materials Control* of this manual.

SECTION 547 – SHEAR CONNECTORS

547.1 – GENERAL

The Standard Specifications contain sufficient information to inspect and test the installation of shear connectors. Spirals and structural shapes are usually shop installed, and since they are not used very frequently in New Hampshire, they will not be discussed.

547.2 – MATERIALS

At an early date, the Contract Administrator should call the Contractor's attention to the requirements that must be fulfilled before ordering the studs as specified in *Subsection 547.2* of the Standard Specifications. Before placement, the Contract Administrator should inspect the studs, making sure that they are the proper size, length, and diameter. If the steel camber is near the tolerance limit, check with the Bridge Design Engineer or the District Construction Engineer for a possible change in stud length.

Figure 500 – 54: Shear Connectors

547.3 – CONSTRUCTION OPERATIONS

- Layout

  The Contract Administrator should check the Contractor's layout prior to the field installation of shear connectors.

- Workmanship

  The studs should be welded on the designated layout points. A maximum variation of 1 in will be accepted, provided the adjacent studs are not closer than 2 ½ in O.C.

- Work Area

  All reinforcing steel, mesh, and other materials and equipment that will interfere with the stud installation shall be removed.

- Safety

  The Specifications now allow studs to be shop–installed. Note that when they are shop welded, the Contractor must provide suitable staging for the Contract Administrator to safely take necessary elevations.

SECTION 550 – STRUCTURAL STEEL

550.1 – GENERAL

This section covers the broad field of furnishing and constructing the structural steel portion of structures. Steel bridges constitute the most important part of this section; therefore, the following paragraphs apply primarily to steel bridges but also apply, in part, to most metal structures (such as handrails, etc.). The Contract Administrator will have to determine what portions of the following paragraphs apply to some of the minor structures or structural components.

550.2 – MATERIALS

The Contractor shall provide shop drawings of the steel through the District Construction Engineer to the Bridge Design Bureau for approval. The steel is then fabricated from the approved drawings under the inspection of a Department approved testing agency or Inspector.

The Contract Administrator should inspect the steel for damage upon its arrival on the project and report to the Bridge Design Engineer in the Bureau of Bridge Design any damage during transport of structural steel before erection.

Figure 500 – 55: Structural Steel – Bridge Construction

Before erection of the steel and any partial payment, the Contract Administrator should have received the following items:

- Approved shop drawings
- Certificates of Compliance
- Steel Mill Test Reports
- Inspection reports
- Erection Plan by NH Professional Engineer, received for documentation, reviewed by the Contract Administrator and District Construction Engineer for constructability
- Subcontractor approval, if applicable

550.3 – CONSTRUCTION OPERATIONS

A. Bridge Seats

Bearing pedestals are sometimes poured approximately ¼ in lower than the plan elevation. After the bearing pedestals have cured, they should be ground level. Elevations are then taken at each pedestal and the difference between the actual and the plan elevations should be computed. Shims equal to the computed differences are then ordered by the Contractor for use under the bridge shoes.

Sometimes bearing pedestals are finished to grade while placing them and checked with a transit or level during the finishing process. Other times, the pedestals are finished slightly high (⅛ in to ¼ in), then ground down level to the design elevation. If this approach is used, it is important that the whole seat be ground down to avoid any ponding on the bridge seat.

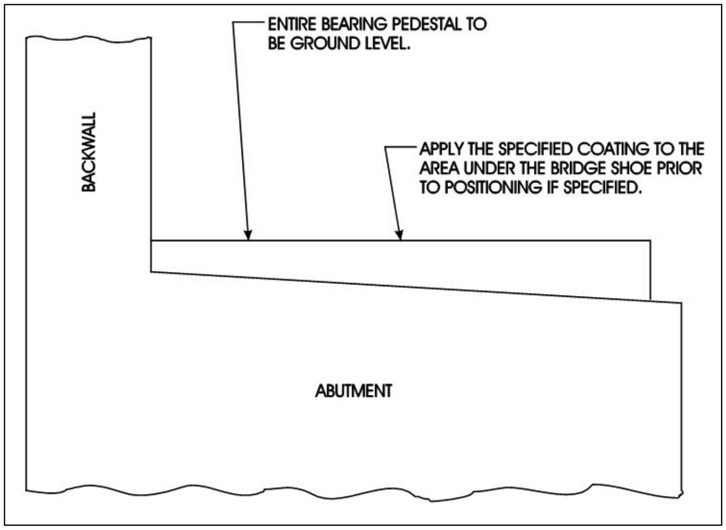


Figure 500 – 56: Leveling Bridge Seats by Grinding

After the bearing pedestals are confirmed level and the elevations are checked, a transit and tape or an electronic coordinate system should be used to accurately locate the centerlines of bearing and girders. NHDOT survey is only obligated to perform an initial layout of the bridge seats. Since this is usually done before the seats are poured, the Contractor is responsible for resetting any and all future points. Regardless of this fact, it is still very important for the Contract Administrator and/or the Inspectors to double check the layout before any bridge shoes or steel are placed.

The Contractor should mark anchor bolt holes for drilling using a bridge shoe template (typically made out of plywood). If anchor bolt holes are to be drilled and left in the winter, they should be filled with antifreeze and covered.

If the application of epoxy coating, water repellent, or concrete sealer is specified on the horizontal bridge seat area, the paint coatings should be applied before the bridge shoes are positioned.

B. Placing the Bridge Shoes

When the Contractor places bridge shoes on the bridge seats, verify that the correct shoe is being used. Different abutment and pier configurations may require different types of shoes, either fixed or expansion, and different girders may have different sized or tapered shoes, together with various sized prefabricated bearing pads for each.

In most cases, shoe anchor bolts should not be grouted until structural steel is completely assembled, bolted up, and accepted. After erection is complete, and just prior to grouting the anchor bolts, the shoes should be positioned for the expected travel due to temperature changes and aligned with the axis of the stringer.

If the rocker type expansion bearings are used, the rockers should be adjusted according to the prevailing temperature so as to be vertical at the standard temperature shown on the plans. In the absence of a plan chart, use ⅛ in for each 10°F of temperature change per 100 lineal feet of bridge. Rockers should be rechecked and reset, if necessary, at various intervals until they all remain in the same relative position. Additional checks will be necessary until the desired results are accomplished. The anchor bolts should not be grouted until all of the rocker adjustments are complete.

C. Storage and Handling of Steel Members

Steel members should be stored off the ground on skids or cribbing. Beams and girders must be placed upright and stored securely so that they will remain upright. Beams and girders are rigid only in the plane of the web and have little resistance to lateral deflection or twisting.

Once the steel has been delivered, the Contract Administrator should inspect it to verify that no paint or protective coating (unless specified) is on the surfaces that are to be in contact with concrete or at steel splice areas. If it is necessary to remove paint in these areas, sand blasting should be employed.

Beam clamps or straps are typically used for hoisting and positioning structural steel. It is prohibited to use pin holes or bolt holes as hook–on points for lifting structural steel. Bolt holes are precisely machined to close tolerances and the material adjacent to a bolt hole can be mutilated when a single hole is used as the hook–on point to lift a heavy member. Tag lines are generally affixed to the member ends to give the Contractor better control over swaying.

D. Steel Erection

Shop drawings contain fabrication code markings, and will provide the Contract Administrator with erection information to identify beams, splice plates, diaphragms, scuppers, end–dam components, and the indicated direction of north or east. The Contract Administrator should be satisfied that the erection procedure will be successful before beginning erection. Any questions should be referred to the Bridge Design Engineer and/or District Construction Engineer.

Factors to be considered for steel erection operations are as follows:

- Crane capacity

- Footings for cranes and out–riggers
- Aspects of cranes working on barges
- Temporary bracing that may be required to support girders after they are erected
- Possible hazards to traffic

During erection, the Contract Administrator should monitor the operation to ensure that equipment use and procedures adhere to the approved–plan. As each girder is placed it should be bolted up and sufficiently braced to keep from collapsing. With the increasing use of curved steel beams, the Contract Administrator should be aware that special erection procedures are necessary. A curved beam, unlike a straight one, will not stand unsupported on the bridge shoes, so the Contractor must modify the erection procedure or use additional falsework.

E. High Strength Bolts

High–strength bolts and nuts may be identified by markings described in *Section 550 Structural Steel* of the Standard Specifications. Bolts, nuts, and washers should arrive on the job with a protective coating of oil, and should be stored and maintained in their original, factory–sealed shipping containers until the day they are to be used. The oil coating prevents the threads, as well as other exposed surfaces, from rusting, which could adversely affect the performance of the fastener during torquing.

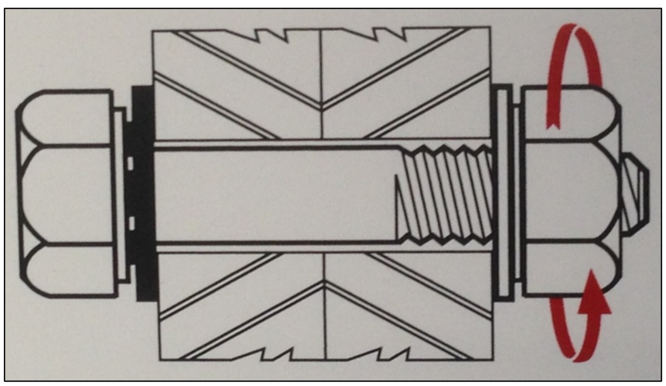


Figure 500 – 57: Typical High–Strength Steel Bolt Assembly with DTIs

When installing high strength steel bolts, there is a direct correlation of between torquing and tensioning. Torquing bolts actually involves achieving tension in the bolt. The bolt threads must be properly lubricated in order to register the actual proper applied tension. Improperly lubricated bolts may register prematurely high torque values because of added friction in rotation, and may even lead to a broken bolt from inadvertent over–tightening.

Without factory or field lubrication, bolts cannot be properly tensioned. The Contractor shall apply additional lubricating wax or oil to all bolt and nut threads as approved by the manufacturer and the Materials and Research Bureau. Due to the fact that final tensioning of all bolts should be done only after all of the structural steel has been erected, it is recommended that the Contractor field lubricate all threads prior to initial installation of a multi–span bridge. This recommendation is based on the assumption that the bolts may sit in structural members for extended periods to time (possibly days, if not weeks) and become weathered, losing their factory–applied lubrication.

When metric bolts are specified, ensure that only metric bolts are used. Because the size of shop–drilled bolt holes are based on the size of the bolt specified, it is very important that only the size and type of bolt specified is actually used. For example, if an M20 bolt is specified, a ¾ in Ø (19.05 mm) bolt may not be substituted, even though the ¾ in Ø bolt is only 1 mm smaller than an M20 bolt. The holes in the plates are sized for a 20 mm Ø bolt, not a 19 mm Ø bolt.

- Proper Tightening of High–Strength Bolts

The Standard Specifications require the use of Direct Tension Indicator washers (DTIs). These washers have bumps machined into them that will compress once the proper bolt tension has been obtained. The Contract Administrator or their assistant to shall check the area under the DTI with a feeler gage to verify that the DTI has been properly compressed without over–compression. Refer to *Subsection 550.2.4.2.4 Direct Tension Indicators (DTI)* of the Standard Specifications to determine the required amount of DTI compression.

Figure 500 – 58: Direct Tension Indicator Washers

When using DTIs, care should be taken when field verifying or testing each rotational–capacity lot of bolted connections, employing the Skidmore. The Inspector shall assess the behavior of both the DTI and the bolt assembly when the connection is brought to both the verification tension and to the "no entries but visible gap" tension.

In certain situations, the Inspector may find it difficult to relate the recommended tension to the appropriate number of entries allowed. Also, it is very difficult to observe a "visible gap" once all entries of the DTI are closed. This is especially true when the DTI is coated or galvanized because the coating may flake off, making the "visible gap" condition undeterminable, thereby making the tension load in the bolt indeterminate.

When encountering issues when relating required tension to the number of entries around the DTI, the Inspector should do the following:

- o Verify the Rockwell Hardness (R_c) of the DTI, which should be less than 35
- o Verify that all bolts, nuts, and washers in the rotational–capacity lot conform to Standard Specifications
- o Verify that the bolt assemblies have adequate lubrication
- o Verify that the bolts and nuts are free of any dirt, burrs, defects, other materials, or any areas of built–up galvanizing that would prevent satisfactory results during verification testing and inspection
- o Verify the satisfactory performance of a particular single connection and measure the bolt reveal or "stick–out" of that connection, using this information to assist in determining the tension in other connections
- o Reject, as a last resort, the DTI lot if all of the requirements set forth in the Standard Specifications were determined to be unsatisfactory during verification testing

Two other methods of tightening listed in the Specifications are calibrated wrench tightening and turn–of–nut tightening. As these methods are considerably less reliable, they should not be used in lieu of DTIs in cases where DTIs are required by the project plans or in the contract Specifications.

Calibrated wrench tightening uses a torque–control wrench that cuts off when a pre–set torque is reached. This method is extremely unreliable if the bolts become rusted or are slightly out of alignment, causing them to rub against the steel splice plates or diaphragms. This can result in a high torque reading but low tension.

Turn–of–nut tightening can be accomplished with either a hand wrench or standard impact wrench, usually powered by compressed air. There must be an adequate pressure at the tool – an absolute minimum of 100 psi for bolts with a ⅞ in diameter and smaller. For larger bolts, higher pressure must be used.

- Calibration for Bolts

No matter what method of bolt tightening is used, the use of a calibrating device to verify DTI and bolt performance is essential. The Standard Specifications detail the necessary testing. The calibrator, furnished by the Contractor, and similar to that shown in the following figure, must be inspected and approved by the Materials and Research Bureau prior to its use on the project.

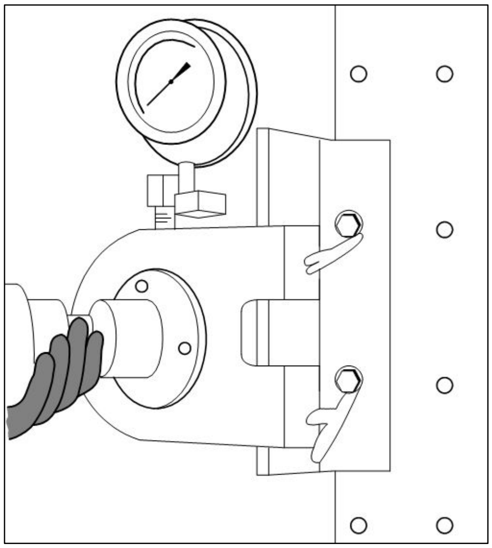


Figure 500 – 59: Bolt–Tension Calibrator (Skidmore)

The bolt–tension calibrator, also known as a "Skidmore," is a hydraulic–loaded cell that measures bolt tension created by tightening. As the bolt or nut is turned, the internal bolt tension or clamping force is transmitted through the hydraulic fluid to a pressure gauge which indicates bolt tension directly. Older bolt–tension calibrators indicate tension in pounds; newer calibrators are dual dimensioned, this is, reporting the bolt tension in both pounds and newtons. The dial of the gauge may be marked to show the required minimum tension for each bolt diameter.

When torque–control wrenches are used, the Contract Administrator must insist that they be calibrated at least once each day by tightening not less than three bolts of each diameter from the bolts to be installed. The test bolts are not to be used for installation and must be discarded. The average torque determined by this calibration procedure may then be used to pre–set the cut–off device built into the torque–control wrench. The torque–control device must be set to provide a bolt tension that is 5% to 10% in excess of the minimum bolt tension.

The bolt–tension calibrator is also necessary to calibrate the hand–indicator torque wrenches, also to be furnished by the Contractor, and is used by Inspectors for checking torque as it relates to tension after tightening by either the calibrated wrench method or the turn–of–the–nut method. Again, if either of these methods is used, a close examination of the condition of the bolts must be performed regularly. Any rust or imperfections can significantly impact the torque–tension relationship.

- Installation Sequence

Regardless of the method used to tighten high–strength structural bolts, the sequence of operations is basically the same. First, holes in splice plates and beams are "faired up" with enough drift pins to maintain dimensions and a plumb alignment. Next, sufficient high–strength bolts of the proper size are installed to hold the connection in place. Only hand tightening is required at this point.

Since these bolts will remain in place as permanent fasteners, washers, if required, should be installed with the bolts during fitting–up. The balance of the holes should then be filled with bolts and assembled with nuts and washers. Fascia beam splice connection bolts should be inserted with the bolt heads exposed to view, with nuts in or up. The test bolts may not be used for installation and must be discarded.

At this time, elevations should be taken on top of the beams at splice connections to determine whether adjustments of beam elevations are needed. Correction of these elevations will keep blocking distance variations to a minimum.

Even with some bolts and drift pins installed, splice elevations may be lifted and then tightened in place or they may be lowered by loosening and lifting the adjacent splice. Desired elevations can be substantially obtained by this procedure.

The following figure shows beam splice elevation parameters.

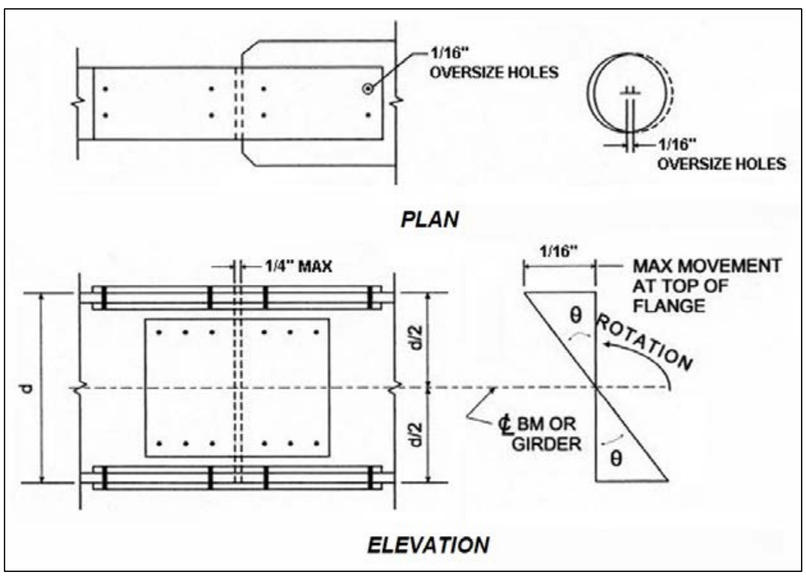


Figure 500 – 60: Beam Splice Elevation Parameters

Calculate the beam splice elevation using the following procedure.

Given:

d = Depth of Beam (in) = 40 in

L = Length of Beam (ft) = 70 ft

ϴ = Beam Rotation Angle

Find:

X = Change in Beam Splice Elevation

Calculate:

TAN ϴ = X / L

and

X = L / 8d

X = (70 ft) (12 in/ft) / 8 (40 in)

X = 840 in / 320 in

X = 2.625 in = 2 ⅝ in

Therefore, the change in beam splice elevation is 2 ⅝ in.

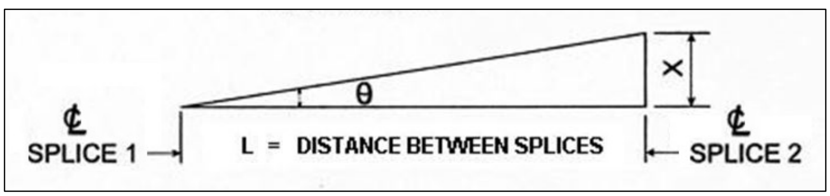


Figure 500 – 61: Calculating the Change in Beam Splice Elevation

Next, the bolting crew should start to "snug" the bolts and nuts. "Snug" is defined as the point at which the air wrench begins to impact solidly. A bolt may be snugged–up by a worker using a spud wrench. When the crew completes the "snugging" of the entire connection, the gap between the plates should have entirely disappeared.

Now the drift pins are knocked out, and the remaining holes are filled with bolts and turned to "snug." The connection is now ready for final tensioning. Bolts and nuts should always be tightened progressively away from the center, the fixed or the rigid points to the free edges.

The following figure shows an example of a suggested tightening sequence for a bolted connection.

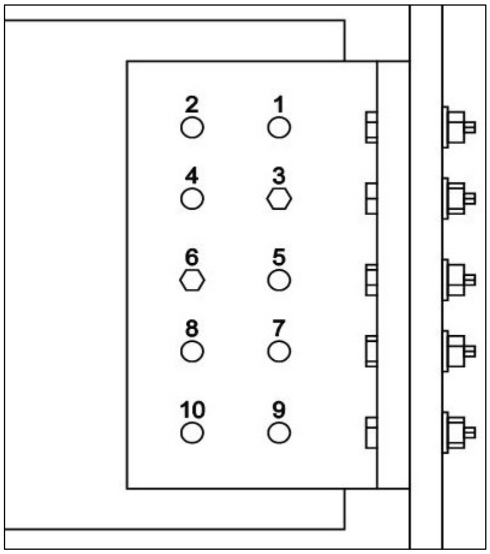


Figure 500 – 62: Example of Suggested Bolt Tightening Sequence at Bolted Connection

Final tightening with a torque–control wrench that has been properly calibrated can proceed in a straight–forward manner. For the turn–of–the–nut method, a hand–wrench is used to hold the end not being torqued to ensure that the true required turn measurement is not lost.

Close examination of a nut after final tensioning will disclose slight burrs or peening marks near the edge of each nut "flat." These marks are caused by the "hammering" action of the wrench as it impacts the nut. If nuts on *ASTM* A325M bolts show no such markings, a thorough inspection should be made to insure that the bolts were properly tensioned. Nuts furnished with ASTM A490M bolts may not show any distortion because of their greater hardness. However, a slight burnishing of the edges should be evident.

- Characteristics of High–Strength Bolts from Tests

Tensile Strength: Mechanical properties for *ASTM* A325M and ASTM A490M high–strength bolts are determined by applying axial tension loading, either to a full–size bolt or to a reduced section machined from a bolt shank. In normal use, however, the tightening operation induces torsional stress as well as internal tension, and laboratory tests have indicated that the bolts' direct tensile strength is reduced by as much as 5% to 30% when tension is induced by torquing. However, it has been proven that high–strength bolts torqued to Specifications below failure retain initial ultimate tensile strength once connected.

Shear Strength: Research studies have also been undertaken to determine how internal bolt tension affects the ultimate shear strength of the bolt. High–strength bolts were torqued to "snug," one half turn from snug, and one–and–a–half turns from "snug" and then loaded in direct shear. The initial preload has little influence on shear strength. Bolts which were severely torqued (one–and–a–half turns) had approximately the same shear strength as those with little pre–load ("snugged") or nearly maximum pre–load (one half turn).

Bolt Clamping Force: The clamping force induced in a high–strength bolt by tightening can be measured by bolt elongation. A calibration curve is used to convert measured elongations to bolt tension or clamping force. Tests have proven that all bolts develop clamping forces in excess of the specified minimum value, and clamping force values are consistent even when elongations vary over a wide range.

Moment Connections: Behavior of high–strength bolts in direct tension or combined tension and shear is particularly important in moment connections. End–plate connections are a typical example. When high–strength bolts are pre–loaded in moment connections, the applied moments normally cause little or no increase in bolt tension, especially if the initial pre–load is well above proof load.

Since external tensile loads merely change the contact pressure, the bolt does not elongate and cause additional bolt tension. However, additional bolt tension may result from prying action due to distortion of the connection details, depending on the relative stiffness of the connected material and the spacing of the fasteners.

Types of High–Strength Bolt Connections: For high strength bolted connections, NHDOT usually requires washers. In the absence of washers, it is difficult to inspect with torque wrenches. The following figure shows different types of high–strength bolted connections.

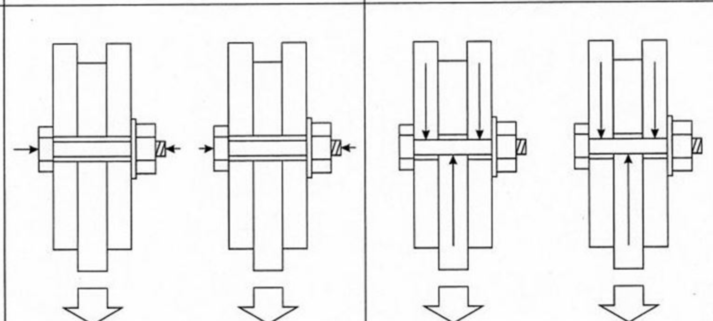
CONNECTION	FRICTION-TYPE		BEARING-TYPE	
JOINT PLACEMENT REQUIREMENTS				
BOLT	A 325 (A 325M)	A 490 (A 490M)	A 325 (A 325M)	A 490 (A 490M)
NUT	A 563 (A 563M)	A 563 (A 563M)	A 563 (A 563M)	A 563 (A 563M)
WASHER - ASTM F 436 (F 436M) (TURN-OF-NUT METHOD) OR (CALIBRATED WRENCH METHOD)	ONE	ONE (UNDER TURNED ELEMENT IF Y. P. IS 40,000 PSI (276 MPa) OR GREATER) TWO (IF Y. P. IS LESS THAN 40,000 PSI (276 MPa))	ONE	ONE (UNDER TURNED ELEMENT IF Y. P. IS 40,000 PSI (276 MPa) OR GREATER) TWO (IF Y. P. IS LESS THAN 40,000 PSI (276 MPa))
THREAD LOCATION	MAY BE IN SHEAR PLANE	MAY BE IN SHEAR PLANE	MUST BE OUT OF SHEAR PLANE WHEN HIGHER ALLOWABLE VALUE FOR BEARING-TYPE CONNECTION IS USED.	MUST BE OUT OF SHEAR PLANE WHEN HIGHER ALLOWABLE VALUE FOR BEARING-TYPE CONNECTION IS USED.

Figure 500 – 63: High–Strength Bolted Connection Examples

Note: The Yield Point (YP) refers to that of the material being joined

F. Welding

If the Contractor requests to field weld a connection not shown as a field weld on the plans, then the Bridge Design Engineer and/or District Construction Engineer must give approval for each field weld requested. Field welding shall meet the requirements of *Subsection 550.3.16 Field Welding* of the Standard Specifications.

Welders shall have passed AASHTO welding specification qualification tests. The minimum qualifications shall be for “all–position” groove welding on a “limited thickness” (¾ in max.) plate. The welder must furnish acceptable proof of these qualifications, namely AWS Certification, sworn copies of a satisfactory qualification test record, or both.

Figure 500 – 64: Shop Welding a Structural Steel Member

More information regarding types of welding, welder qualifications, and welding symbols may be found on the American Welding Society website at the following URL: www.aws.org

- Welding Inspection and Troubleshooting: Use the following troubleshooting charts as an aid in welding inspection.

Welding Troubleshooting Chart (1)		
Welding Problem	Cause	Solution
Shallow Penetration	Welding speed too fast	Use speeds recommended in the procedure tables.
	Electrode too large	Use smaller electrodes for deeper grooves.
	Current too low	Use an adequate current for deep penetration.
	Faulty preparation	Allow for some gaps (free space) at the bottom of the joint.
Distortion or Warping	Uneven heating of part	Preheat before welding; Post heat after welding; Make small welds at several points on the part to distribute heat evenly; Weld in solid bars or rounds where a buildup is required; Back step, welding intermittently; Use fewer passes where possible.
	Welding wrong area	Perform all forming operations prior to welding; Do not over–weld; Assure proper edge preparation and fit–up.
	Shrinkage of weld bead	Tack or clamp parts together properly in rigid fixtures; Pre–form parts to counteract distortion; Peen the bead when it cools.

Welding Troubleshooting Chart (2)		
Welding Problem	Cause	Solution
Cracking	Base metal is not within the recommended analysis range	Use a less penetrating electrode or decrease the current; Change the steel analysis; Shorten the arc length to make the beads more convex; Decrease the welding current and use more passes; Decrease the welding speed; Use a low–hydrogen type electrode, such as the *Jetweld LH–70*; Back step to penetrate the previous beads, thereby reducing the concentration of undesirable elements in the weld crater; Preheat the parts before welding.
	Improper joint preparation	Leave a 1/32 in gap between plates for free movement; Position and weld slightly upwards (about 5°) to increase the weld section on the first pass.
	Weld joint is too rigid	Weld towards the unrestrained end of weld joints.
	Welds too small or wrong shape	Change to a less penetrating type of electrode; Decrease the weld current, using more passes.
Undercutting	Welding current too high	Correct the current and travel speed; Decrease the electrode size.
	Faulty electrode manipulation for job	Weld slowly, using a lower current; Avoid excessive weaving, but use a uniform weave in butt welding.

Welding Troubleshooting Chart (3)		
Welding Problem	Cause	Solution
Surface Holes and Porosity	Low carbon in base metal	Verify that the carbon and manganese content are sufficiently high and the sulfur, phosphorous, and silicon content are sufficiently low.
	Poor electrode selection	Change to a low–hydrogen type of electrode, such as the *Jetweld LH–70*.
	Insufficient puddling	Allow a sufficient puddling time for gases to escape.
	Excessive arcing	Decrease the current, using a shorter arc.
	Steel is outside recommended analysis range	See “Cracking”
Arc Blow	Stray magnetic fields cause arc to deviate from its intended course when welding with DC	Switch to AC welding if at all practical (the best solution); Shift the ground clamp to the other end of the work; Hold as short an arc as possible; Change to a smaller electrode; Use a lower welding current; Weld to a heavy tack, or a weld already made; Weld in the same direction as arc blow; Use steel blocks to alter the current path around the arc; Change to an alternate electrode with greater arc force; Use back–stepping on long welds; Tack a small plate across the seam at the weld end; Use run–out tabs; Ground the work at several spots.

Welding Troubleshooting Chart (4)		
Welding Problem	Cause	Solution
Weld Spatter **Note**: Although weld spatter does not affect structural strength, excessive spatter has a poor appearance and will increase cleaning costs	Current too high	Check the current settings for the electrode size and thickness used.
	Wrong electrode choice; Wrong polarity	Use an electrode with minimum–spatter characteristics; employ correct polarity for electrode.
	Too large an electrode	Use an electrode with the correct diameter.
	Wrong electrode angle	Use the electrode angle recommended in the procedures tables.
	Arc blow	Apply corrections for arc blow; verify that the electrodes are perfectly dry.
Poor Fusion	Current setting too low	Verify that the proper current is being used; Use the stringer bead technique.
	Dirt and ragged edges in joint	Verify that the weld is clean and the sides of the joint are smooth.
	Welding electrodes too large for groove or fillet	Use smaller electrodes or an alternate electrode; change joint preparation.
Rough Welding	Moisture pick–up	Store electrodes in a cabinet or room that is about 10°F warmer than the surrounding atmosphere; if electrodes have become damp, dry them at 200°F for one hour; low hydrogen electrodes require a range of drying temperatures from 300°F to 700°F.

Welding Symbols

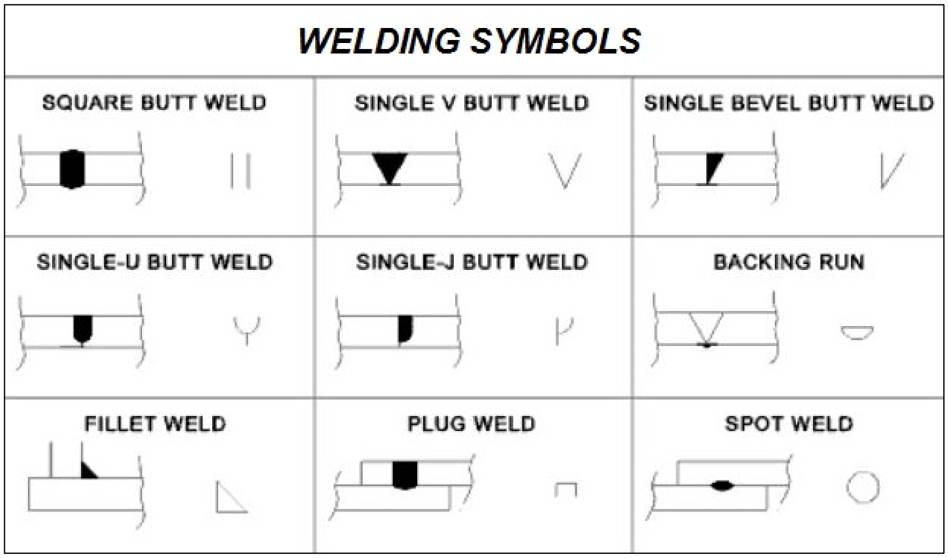


Figure 500 – 65: Welding Symbols

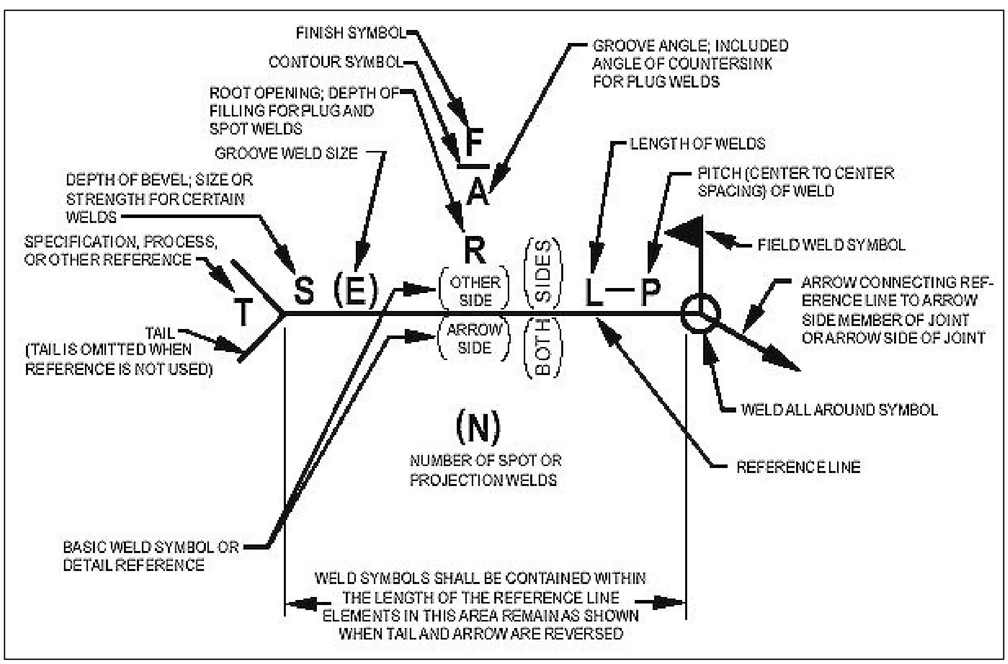


Figure 500 – 66: Welding Diagram

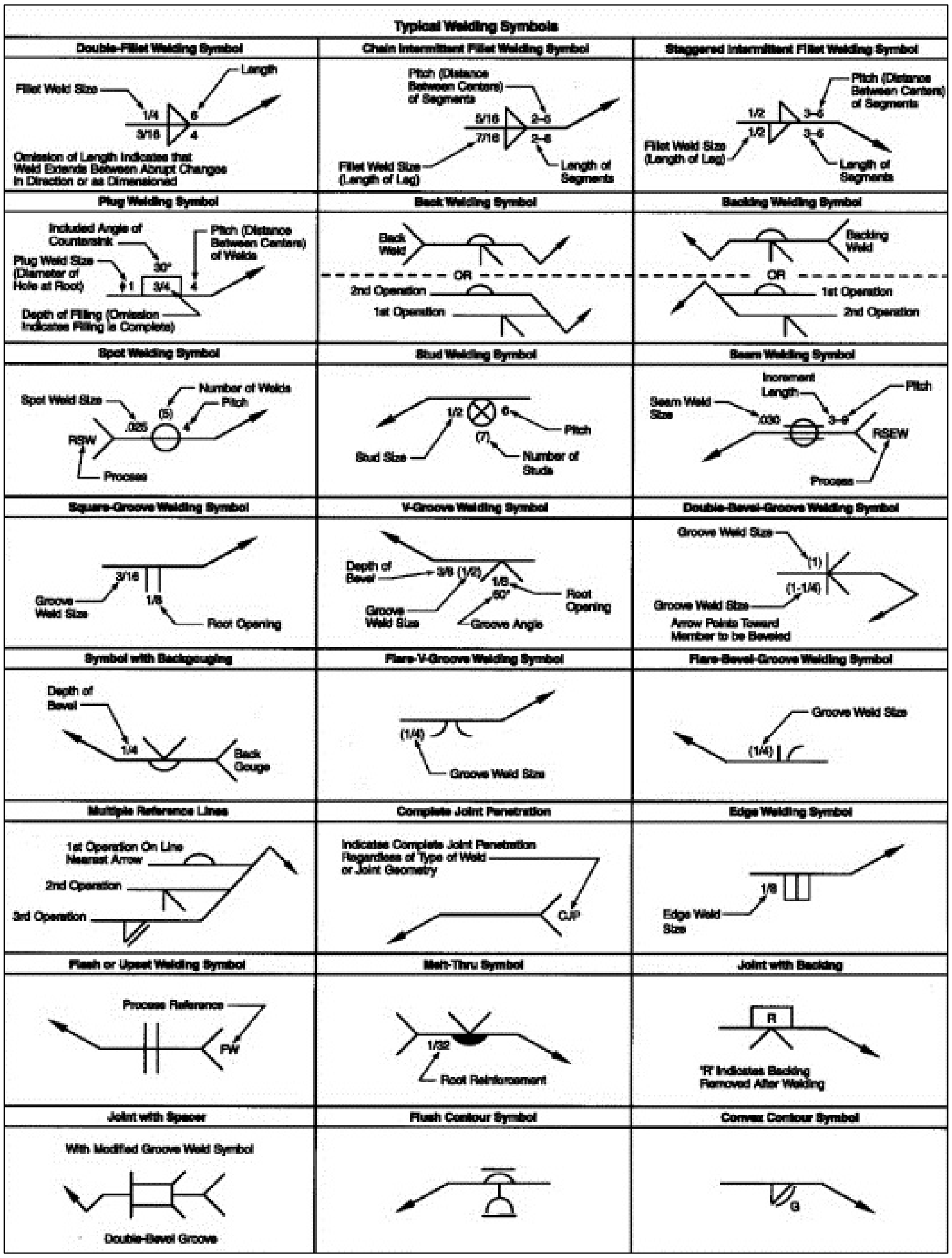


Figure 600 – 67: Welding Symbols for Specific Applications

G. Camber and Haunch Grades

The vertical alignment of a bridge deck is defined by a profile grade line in the same manner that a roadway grade is defined. A bridge deck generally consists of a concrete slab that must be built correctly to grade. All girders should be checked for camber as soon as they are erected, with diaphragms and splices completed, by shooting elevations at beam locations shown on the plan deflection diagram (generally the 10th points between piers and/or abutments.

If a girder shows an excessive amount of camber (plus or minus) that cannot be attributed to and corrected by erection splicing (see method of adjusting splices previously discussed under bolt tensioning), adjustments can be computed into the haunch grades. Where the computed haunch grades would indicate less than a zero haunch, a haunch varying much from the typical shown, or a haunch so high that the shear connectors are not in the lower mat of reinforcing steel, the Bridge Design Engineer and the District Construction Engineer should be consulted.

If the above goes unattended on single span bridges, a "hump" in the bridge rail may be very distinct, particularly if the bridge has wings parallel to the deck and the same line of rail runs from the deck through the wings.

H. Sandblasting

Prior to painting, all steel must be sandblasted clean either by the brush off, commercial, or near white method. These methods are determined by the degree of deterioration or corrosion. The Contractor shall furnish information from the Society for Protective Coatings (SSPC) with photos of various degrees of sandblasting. This will be provided through the contract.

Information regarding sandblasting structural steel may be found at the Society for Protective Coatings website at the following URL:

http://www.sspc.org/

Important: If old lead based paint is being removed, it is very important to hold preliminary meetings with the Contractor to discuss safety, access, containment, toxic disposal, etc.

I. Painting

The Contractor should submit a 5 gal. can of each type of paint to the Bureau of Materials and Research prior to painting. Where more than one lot number is to be used, each lot is to be submitted for testing. As previously mentioned, paint found on contact surfaces should be removed by sandblasting prior to erection. Surfaces that will be inaccessible after erection will have to be painted with the full number of coats prior to erection. The CA should ensure that the epoxy paint systems ordered on the plans are applied.

J. Steel Erection Checklist

The following steel erection checklist should be used as a "memory–jogger" for conducting steel erection operations.

- Approved shop drawings
- Certificate of Compliance
- Mill test reports

- Inspection reports
- Erection plans
- Subcontractor approval

  Pre–erection meeting:

  - Schedule
  - Time frame of installation
  - Weather considerations
  - Public Awareness
  - Set–ups
  - Challenges
  - Designated traffic control person
  - Designated safety officer
  - Field change procedures
- Press release
- Contact the Transport Management Center (TMC) at (603) 271–6862
- Contact Police, Fire, and Maintenance
- Emergency contact list
- Bridge shoes/assemblies/bridge seats
- Verify seat elevations prior to grinding
- Verify seat elevations after grinding
- Drill anchor bolt holes prior to setting structural steel

Note: Bolt holes may need to be filled with antifreeze if below freezing temperatures are anticipated along with the potential for precipitation; bolt holes should be prevented from filling with water that could freeze.

- Concrete bearing seat
  - Mark the center of bearing on the left and right sides of seat
  - Mark the center of the girder/sole plate on the front and back of seats
- Bearing pad
  - Mark the centerline left to right and front to back
- Masonry plate

  - Longitudinal arrows should point up station
  - Mark the center of masonry plate left to right and front to back
  - Mark the guidelines for the bottom of the girder flange's left and right edges
- Check pad/level to pad/mark layout for placement
  - Verify that assemblies are placed properly
  - Verify the centerlines on the bearing masonry/sole plate edges
- Drill anchor bolt holes prior to setting structural steel

  Note: Bolt holes may need to be filled with antifreeze if below freezing temperatures are anticipated along with the potential for precipitation; bolt holes should be prevented from filling with water that could freeze.

- Apply 534.x water repellant
- Inspect all materials, coatings, and welds for compliance
- Identify all assemblies and stage them in order of placement
- Mark the girder flanges position
- Verify if bearings need to be tapered
- Use shims for height adjustment as necessary
- Shoring Towers (if required)
  - Check pad/stability
  - Set to total camber allowed to deflect to dead loading
  - Verify elevations
- Girders
  - Inspect girders for compliance
  - Stage girders in order of placement
  - Identify each girder
  - Mark each girder number, e.g., 1.1, 1.2, 1.3…, 2.1, 2.2, 2.3…
  - Mark girders placement points on Pier and Abutments
  - Verify the crane's location identified for lift
  - Beam Rigging
  - Mark the center of gravity and clamp points
  - Verify whether slope is being set for lift
  - Verify the length/weight/angle of each girder prior to lift

  - Verify that girders are being set on wood to prevent slide out from bearing
  - Mark girders for quick cross–frame identification/connection
  - Verify shims requirement
  - Verify beam camber and adjust prior to final tightening
- Temporary Girder Support
  - During erection operations, it is necessary to support the first placed girder until the second girder is installed and the two are stabilized with diaphragms between them
- Utility Lines
- Weather conditions– Rain, snow, ice, wind, etc.
- Splices
  - Verify the initial/final DTI (Direct Tension Indicators) percentage values
  - Verify method for multiple splices
- Cross frames/diaphragms
  - Inspect cross frames/diaphragms for compliance
  - Stage cross frames/diaphragms in order of placement
  - Identify and mark each cross frame/diaphragm for quick girder connect
  - Verify the initial/final DTI (Direct Tension Indicators) percentage values
  - Ice Issue – Heat to prevent false kip reading/ice
- Bolts/nuts/DTIs
- Using a Skidmore, Test Nuts/Bolts/ DTIs
  - Verify that the Skidmore has been calibrated within the last year
  - Calibration papers from approved lab
- Identify bolts to be used
- Verify correct size feeler gages
- Verify bolt size, initial/final kips, and penetration
- Confirm Markings on Bolts/Nuts/DTIs
  - Verify that all of the bolts are from the same lot
  - Certificate/paperwork for the rotational capacity test
- Properly store all bolts/nuts/DTIs and keep them dry
- Apply paraffin treatment stick wax in lieu of sprays

- Submit all required documentation to the Materials and Research Bureau
- Cranes
    - Conduct lift capacity calculations
    - Verify placement operation
    - Verify that pads are properly positioned
    - Monitor crane mobilization
    - Monitor crane staging
    - Monitor crane set–ups each day
- Traffic Control
    - Traffic Delineation
    - Traffic Delay Time – A.D.T. Count
    - Lighting
    - Message Boards
    - Officers/Flaggers Lined Up, Communication During Operation
    - Contact Local Police, Fire, Rescue, etc…
    - Emergency Contact List

SECTION 562 – SILICONE JOINT SEALANT

562.1 – GENERAL

Silicone joint sealant is a silicone rubber compound with elastic properties designed to give a tightly sealed joint, and is most generally used in expansion joints, such as between bridge wingwalls and abutments.

562.1 – MATERIALS

The Standard Specifications adequately describe material to be used for the type of joint to be sealed. Unless otherwise specified, the color of the mixed sealing compound should be that which blends with adjacent surfaces. For joints that move in a shearing direction (i.e., deck to pilaster) specific materials shall be used as directed by the Materials and Research Bureau.

All sealants used must be on the Qualified Products List. A Certificate of Compliance from the manufacturer must also be obtained, and labeling on cans should be examined for correct contents before use.

Figure 500 – 68: Silicone Joint Sealant Application in Expansion Joint

562.3 – CONSTRUCTION OPERATIONS

After proper application of the sealant, as described in the Standard Specifications, the sealing compound should maintain the properties of a tack–free, cured, rubber–like material that protects the joint against the infiltration of water and other foreign matter. Application of all Silicone sealants should be completed prior to application of other concrete sealers and waterproofers.

SECTION 563 – BRIDGE RAILINGS

563.1 – GENERAL

Bridge railings add safety and beauty to a bridge. Proper installation of bridge railings requires close monitoring by the Contract Administrator in the layout of rail anchor bolts and the alignment of the erected railings.

Figure 500 – 69: Bridge Railing Installation

563.3 – CONSTRUCTION OPERATIONS

Anchor bolt spacing as shown on the plans should be verified before any layout work is done. Since sidewalk concrete is often placed in intermediate sections, an error in one section will affect the spacing across the entire structure. Anchor bolt layout for the entire structure should be field checked before permanently setting any one section.

Anchor bolt units should be set carefully with the aid of a template and secured firmly to prevent movement during concrete placement. However, welding extra reinforcing steel to the anchor bolts to hold them in place is prohibited. It is critical that the reinforcing steel and anchor bolt assemblies be installed per plan, that is, no cutting of or welding on the coping steel.

Any cutting or welding that is not approved may weaken the railing and coping system so that it no longer withstands crash test requirements. In recent years, bridge copings have been widened to allow easier installation of the anchor bolts assemblies within the coping reinforcing steel.

The exposed length of anchor bolts above the sidewalk concrete should be computed by adding the dimensions of the preformed bearing pad, post base, washer, nut, and exposed thread, and checking for conformity with the plans and shop drawings. As a good practice, the Contractor should tape or cover the exposed threads prior to placing the sidewalk concrete to minimize the amount of cleaning necessary to install the nuts.

When the posts are erected, the bearing surfaces should be in full contact. Ensure that sufficient shims have been fixed in position to align rail posts as specified on the plans, either plumb or normal to grade. Verify that all hardware elements on steel or aluminum railing are affixed properly. Final adjustments to the railing should present a pleasing line and grade.

On bridges where the rail is continuous through wings and span, a check should be made of the rail location as soon as the deck is poured to verify that the expected dead load deflection has been substantially realized. If it has not, some adjustment will be necessary to carry the excess camber through the full length of the rail; otherwise, a hump in the span portion will be obvious.

For a relatively small amount of correction where no part of the wings have already been poured to grade, adjustments to the curb and sidewalk elevations may be made to distribute the excess height at the center of the span through the full length of rail making the hump long, smooth, and considerably less objectionable. If the curb and sidewalk cannot be sufficiently adjusted, the second solution is to raise some of the posts to provide a pleasing appearance to the entire rail. This can be done by raising some of the anchor bolts and by either shimming or grouting under the posts. The Contract Administrator dealing with these issues should discuss these possible solutions and the specific details with their District Construction Engineer.

SECTION 564 – BRIDGE LIGHTING SYSTEM

564.1 – GENERAL

The bridge lighting system includes anchor bolts, junction boxes, fittings, and conduits that are a permanent part of the bridge structure. Proper installation of the system will allow the local electric utility to erect light poles and wire lights after the bridge construction is completed.

564.2 – MATERIALS

All materials used under this specification must be covered by Certificates of Compliance.

564.3 – CONSTRUCTION OPERATIONS

- Coordination with Utilities

  Before the Contractor commences any work associated with the lighting system, they should check with the supplying electric utility to determine whether sizes of conduits, junction boxes, design of fittings, and the number, sizes, and locations of anchor bolts are correct. This has usually been done in the design stage but experience has shown that the utility sometimes has a change of policy or equipment between the time of design and actual construction that will affect the system.

- Anchor Bolts

  The Contract Administrator should review the plans to see whether the anchor bolts for the light standards are incorporated in the deck concrete pour or just the sidewalk/curb pour. Also, the location and thread reveal should be checked just before pouring concrete and just after finishing.

- Bearing Surfaces

  If aluminum light pole standards are to be used on concrete bridges, the contact surface of the base should be painted with asphaltic paint or separated from the concrete by a suitable pad.

- Conduit and Junction Boxes

The conduit system should be completed and satisfactorily anchored before concrete is poured. At the ends of the bridge, be sure that conduit is placed before the approach slabs are poured and extends well beyond the slab so it can be connected to later.

SECTION 566 – ELASTOMERIC BRIDGE JOINT SEAL

566.1 – GENERAL

Elastomeric bridge joint seal is normally used in expansion dams, in sidewalks, and between parallel bridge decks where a water–tight joint is required.

566.3 – CONSTRUCTION OPERATIONS

The plans should be checked and the joint seal manufacturer's instructions carefully followed throughout the joint seal installation process. For a properly–configured, water–tight joint, all of the end dam steel surfaces should be cleaned of any dirt, oil, or concrete spatter before installation of the neoprene. The elastomeric joint seal itself should be clean of any dirt, oil, or dust for proper bonding of adhesive lubricant. Under some conditions, the seal must also be field–cleaned with an approved solvent immediately before application of additional materials.

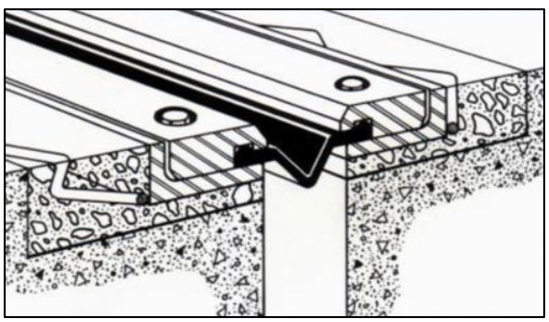


Figure 500 – 70: Elastomeric Bridge Joint Seal Diagram

Sufficient lubricant adhesive material should be used to ensure that the seal is easy to install with a resulting water–tight joint. The joint should be laid in a smooth S–curve configuration where the roadway and sidewalk meet to achieve the proper depth joint across the sidewalk. Relief cutting of the seal at the S–curve must be carefully executed, avoiding any structural damage to the seal.

In general, joint seal installation should be one of the last operations in the bridge construction process and preferably after the roadway has been paved. No vehicular traffic of any kind should be allowed to drive over the newly–installed joint seals seal prior to paving operations to avoid any damage to the seal.

SECTION 568 – STRUCTURAL TIMBER

568.1 – GENERAL

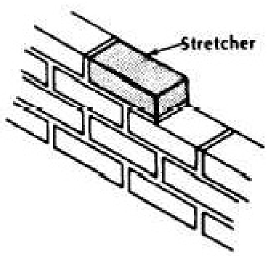
Structural timber is rarely used on current NHDOT projects, but is covered in the Standard Specifications.

SECTION 570 – STONE MASONRY

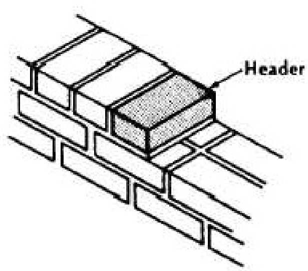
570.1 – GENERAL

Stone Masonry is covered in the Standard Specifications. However, some terminology may be unfamiliar to the some Inspectors. The following glossary defines some of these terms:

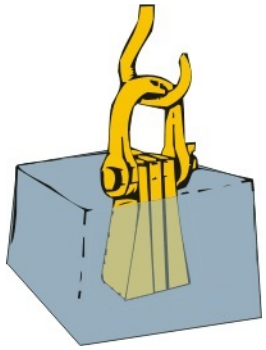
- Stretcher: A stone with its long axis laid parallel to the face of the wall



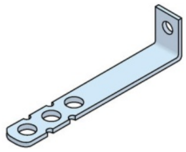
- Header: A stone with its end toward the face of the wall



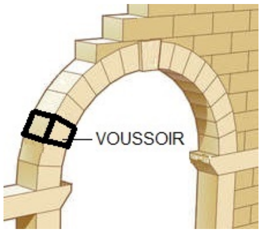
- Lewis: An iron dovetailed tenon, made in sections, which can be fitted into a dovetail mortise to hoist stones



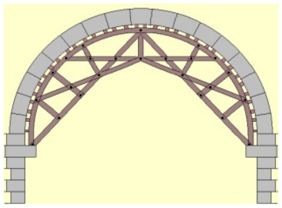
- Cramp: A device, usually of iron bent at the ends, used to anchor mortared blocks of stone



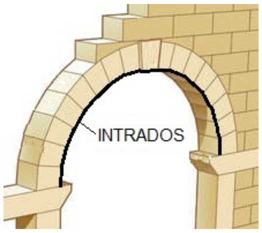
- Voussoir: Any of the wedge–shaped pieces of which an arch or vault is composed



- Centering: Falsework for a masonry arch



- Intrados: The interior curve of an arch



- Extrados: The exterior curve of an arch

Note: Efflorescence is one major issue with stone masonry, as it is with poured concrete. Refer to *Subsection 520.3(S) Efflorescence* for more information about efflorescence and recommended ways to remove it.

SECTION 582 – SLOPE PAVING WITH CONCRETE

582.1 – GENERAL

Slope paving is primarily constructed at structures, and at times in other locations. Its purpose is to give an aesthetic appearance, as well as to stabilize slopes that cannot otherwise support plant growth for stabilization.

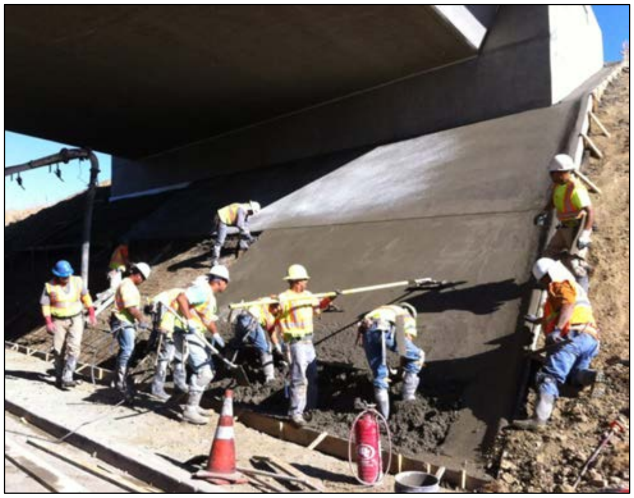


Figure 500 – 71: Concrete Slope Paving

582.2 – MATERIALS

Description of materials to be used for slope paving is adequately given in the Standard Specifications.

582.3 – CONSTRUCTION OPERATIONS

Although the Standard Specifications give a good description of what the finished product should look like, the plans should be checked to determine whether or not additional detail is required. Extra depths of bedding or a gravel course below the bedding may be called for in some instances.

Fine grading of the bedding is a must to provide a well–aligned and uniform appearing product. In most cases, the bedding can be formed, graded and screeded in preparation for laying the slope paving.

SECTION 583 – RIPRAP

583.1 – GENERAL

Riprap protects embankments, dikes, and channels from eroding under the effects of relatively low–velocity water. Riprap provides relatively smooth, tight protection against wash and scour, whereas stone fill is large and rough, intended to dissipate the energy of high–velocity turbulence.

Figure 500 – 72: Riprap Installation

583.2 – MATERIALS

In addition to requirements stated in the Standard Specifications, the Contract Administrator should demand uniformity in the type of stone used and the finished appearance. If the Contractor begins with quarry stone, there should be enough stone to complete the entire installation. In addition, if a gravel blanket is not shown on the plans, the Contract Administrator should check to ensure that the subgrade material is satisfactory to support the stone and is not of a gradation that will be eroded by water circulating through the riprap.

583.3 – CONSTRUCTION OPERATIONS

- Field Check

The Contract Administrator should thoroughly review the plans for locating the riprap and note any field conditions that require adjustments in the preliminary design. The riprap installation should be located in proper respect to high water elevation; direction of flow and angle of impingement; type and security of trees and vegetation; and any springs or drainage water courses that might affect the stability of the design. The installation should also be blended into already stable areas.

- Layout

The Contract Administrator should approve the Contractor's layout before stone work is begun and should verify not only the plan location, but also the subgrade and riprap elevations. The Contractor must exercise care not to disturb the existing ground adjacent to the riprap installation.

- Placing Stone

Riprap installed properly should be carefully supervised, with workers choosing the best face of the stones to expose and setting each stone to grade. Only clean stone should be used and the subgrade must be even and well–compacted before placing any stone.

- Inspection

The Inspector should check the size of the stone to insure conformity with the Specifications. The individual stones should be placed in contact with each other. All open joints shall be filled with smaller stones firmly rammed into place.

- Policy

Riprap construction should be coordinated in order to be accomplished when the embankment or channel subgrade is completed and before erosion is allowed to take place.

SECTION 585 – STONE FILL

585.1 – GENERAL

Stone fill will consist of Class A, B, C, or D stone as specified on the plans and described in the Standard Specifications. The purpose of stone fill is to prevent erosion. The area designated to receive stone fill should be studied to see that the plan design is proper and adequate to meet field conditions. The Contract Administrator should recommend to the District Construction Engineer any extension or modification of the stone fill if there is doubt concerning the adequacy of the plan design.

585.2 – MATERIALS

Stone sizes should be verified to conform to requirements for the class specified.

585.3 – CONSTRUCTION OPERATIONS

The construction of stone fill should be accomplished in a timely manner when access is most available. For example, stone fill should be placed and graded in the toe of embankments before the embankment is built to an appreciable height. Also, stone fills in front of abutments should be constructed prior to steel erection.

Batter boards or string lines erected on tall 2x4 lumber stock are usually necessary to obtain proper grading. Batter boards should be marked with grading requirements, for example, “Cut 2 ft to subgrade” or “Cut 1 ft to top of stone fill.” The Contractor should be encouraged to set these types of batter boards.

When a seal of stone spalls is specified or ordered it should consist of uniformly graded rock fragments ranging downward from 1 ft^3 in size. The completed blanket should be made as void–free as possible by working and compacting the spalls.

After the stone fill work has been completed, the Contract Administrator or Inspector should check surface stones to insure that they are chinked and securely locked in place, giving a pleasing finished appearance. If stone fill is placed along a stream likely to be fished, be sure that there are no stones that can be easily dislodged.

SECTION 591 – STRUCTURAL PLATE PIPES, PIPE–ARCHES, AND ARCHES

591.1 – GENERAL

This item is most often used when an economical structure is needed for short spans, as in small watercourses or passageways such as a cattle pass.

591.2 – MATERIALS

Materials must comply with the Standard Specifications, and Certificates of Compliance must accompany delivery.

591.3 – CONSTRUCTION OPERATIONS

The excavation and bedding requirements are well covered in the Standard Specifications other than in the case of watercourse structures. Verify that gravel, both for bedding and backfill, is the granular backfill material to be used. Sand should not be used, since it may become super–saturated if bed locations are not kept completely dry and proper compaction cannot be obtained. Also, if flash floods or high and fast water conditions occur in a stream, sand may be more subject to undermining and washout.

In most cases, structural plate pipe and pipe arches are preassembled or need only to be bolted together in sections. When a structure is to be assembled in place, detailed erection instructions will be provided. In this case, it is necessary to follow the erection and bolting procedures of the manufacturer and the Standard Specifications.

Figure 500 – 73: Structural Plate Pipe Culvert In–Place Assembly

Usually when a structure is shipped preassembled, the galvanizing is done at the manufacturer's plant, and the Contract Administrator need only check for any bruised or broken places in the spelter coating.

Backfilling is well–covered in the Standard Specifications but it should be stressed that thorough compaction is needed under the haunches of a pipe or pipe–arch, and should be done equally on each side. The strength of the structure, as well as the provision of a stable roadway without future settlement, is dependent upon a well–compacted backfill. It is imperative that a properly formed and compacted bed be used; otherwise, there may be considerable difficulty in bolting up the sections.

SECTION 593 – GEOTEXTILES

593.1 – GENERAL

Refer to the Qualified Products List to determine the type of geotextile required based on its application, strength class, structure, and filter category.

Figure 500 – 74: Geotextiles used for Slope Erosion Control

A Supplemental Specification (January 2004) clarifies when and what type of geotextile should be used. In summary, geotextile "applications" are described based on their most common use, as follows:

- Strength Classes: Geotextile strength classes range from low to extra high strength and are defined by AASHTO *M 288–6 Standard Specifications for Transportation Materials and Methods of Sampling and Testing* or applicable *ASTM* standards.
- Structure: The geotextile structure defines the basic composition of the fabric.
- Filter Categories: The filter category specifies the required permeability and apparent opening size (AOS) properties of the fabric.

The Specifications and Qualified Products List should be consulted to ensure the proper fabric is being used for the required purpose.

The Qualified Products List may found at the following URL:

http://www.nh.gov/dot/org/projectdevelopment/materials/research/products.htm