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STUDY OF OLD BRIDGES FOR WIDENING AND RETROFITTING ON THE BASIS OF WIDENING OF NULLLAH LEI BRIDGE NEAR CHAKLALA

Submitted by

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STUDY OF OLD BRIDGES FOR WIDENING AND RETROFITTING ON THE BASIS OF WIDENING OF NULLLAH LEI BRIDGE NEAR CHAKLALA

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DEDICATION

To my late father who is no longer with me to enjoy these moments and see the fulfillment of his wishes.

To my dearest mother and my family who sacrificed there comfort and interests and motivated, supported and encouraged me in every step and occasion of the course.

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Allah is very kind, merciful and compassionate. His benevolence and blessings have enabled me to accomplish this task.

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ABSTRACT

Many highway bridges become functionally obsolete due to inadequate width long before they become structurally deficient. Since widening almost always is more economical than complete replacement, there is a need to make available the results of research and field experience pertaining to the widening of bridges. The objective is to enable widening of functionally obsolete bridges as needed to eliminate traffic operation and safety problems. Where ever an existing bridge was designed for a specific traffic load and has not deteriorated appreciably, widening is likely to be more cost-effective than complete replacement to accommodate increased traffic load. Guidelines need to be provided for widening/rehabilitation of such bridges. A two lane bridge constructed in 1906 existed on Nullah Lei on Airport road joining Rawalpindi with the Islamabad International Airport. Due to increase in traffic over the period of time the bridge became a bottle-neck to smooth flow of traffic especially during rush hours. The road way of the bridge was extended from 24 ft to 36 ft during 1996. Techniques adopted were studied in detail for extension of other (similar) bridges. It not only eliminates hazards caused by narrow bridges also gives saving of resources/funds along with saving of time. The technique can be applied to existing narrow bridges designed for current traffic loads and having pier/abutment projections in transverse direction.

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Chapter 1
INTRODUCTION

1.1 BACK GROUND

The provision of an effective transport network between the major centers of population, industry and commerce is an essential pre-requisite for economic growth and prosperity in the community as a whole. The unprecedented economic growth, in recent decades, has placed an equally unprecedented demand on all modes of transport including air, rail and road. This demand has led to increasingly heavier volume of traffic on roads, both of which have had, and will continue to have a direct effect on the road structures, especially on bridges. This research principally addresses the subject of widening of bridges. Traffic volume is governed by the capacity of the road, which may be further restricted by the bridges. The development of the Transport Policy for accommodating future traffic growth over bridges is therefore strongly influenced by programs such as that for road widening.

During the last two decades there has been a substantial growth of traffic in Pakistan. Road transport is the main means of moving both goods and people, catering for nearly 90 per cent of all inland needs. Accordingly within the last two decades, traffic on roads has also grown many folds. Main arteries connecting the major cities have witnessed this growth in abundance along with a substantial increase on the internal roads in these cities.

Due to the absence of any data bank, it is extremely difficult to forecast such increases and despite a continuing investment in roads, the result has been an ever-growing traffic congestion. Moreover, though the pivotal position that road transport has in relation to economic growth might be debatable; there is as yet no major change on the horizon that seems likely to upset the dominance of this from of transport. Traffic congestion will increase in parallel with traffic growth in Pakistan unless a sustained and long-term effort is made to overcome the problem. Congestion is extremely inconvenient for all road users, encouraging traffic to divert to less suitable roads, increasing the risk of accidents and reducing safety for the public at large. It also carries an environmental penalty by causing inefficient burning of fuel and an increased level of exhaust emissions. But most importantly, it is a source of increased cost and adversely affects the Pakistani industry, making it less competitive with other countries in world markets. The prospects of growing traffic congestion, spreading throughout the key routes in the national road network is not a desirable situation for any government with over Rs 500 billions invested in the national road network without users comfort being paramount. Accordingly, the whole road programme was re-examined and an expanded road programme was announced adding over 1000 miles of new widening roads to the national road network/motorway and super highway over this decade.

1.2 PURPOSE

The widening of existing highway bridges has become quite common. Several factors contribute to this demand for wider bridges.

1. Increased traffic volumes requiring additional lanes.

2. The need to eliminate hazards caused by narrow bridges.
3. The need for bikeways, pedestrian ways, and wider shoulders.

There are many problems unique to bridge widening that are not encountered during construction of bridges. Failures or serious maintenance problems may result due to lack of proper understanding of these problems. Each bridge widening is unique in itself.

Presently the availability of funds under special programs is enabling public agencies to widen many partially obsolete bridges to eliminate traffic hazards. If an existing bridge designed for low volume of traffic has not deteriorated appreciably, widening is likely to be more cost effective than complete replacement.

Emphasis will be placed on construction practices, but since construction sequence, structure type, framing details and other decisions critical to the success of the work are determined during the design phase, some discussion of design concepts must, of necessity, be included. Structural analysis and design for bridge widening will not be addressed. Much of the discussion that follows can also be applied to new bridges that are constructed in stages, part width at a time.

1.3 NECESSITY

This study is focused on problems arising due to increase in traffic volume over the years, resulting in traffic congestion over the artificial defiles like bridges thus necessitating there widening. The benefits of making extra road provision by virtue of widening have been examined and the various methods of widening have been illustrated. A major issue is that many bridges will have to be replaced when widening is carried out. Bridges are designed to last for 100 years and their replacement, typically only 30 years after they were first built, means that not only is road widening incurring heavy additional costs but traffic disruption also adds to the invisible expenses.

When the original road network was constructed there was no grand perception that widening would ever be needed and under those circumstance as there was no perception that bridges would be replaced in so short a time. In hindsight, that is seen to be shortsightedness and it is now considered appropriate to ensure that bridges are provided with as much flexibility to meet future needs as possible. Cost estimates for alternative designs, including those for bridges, should take into account the capital costs of construction as well as the discounted costs of any demolition and the associated traffic disruption that may occur.

Projected traffic forecasts show that many highways will reach congestion levels again within 30 years of being constructed/widened the period over which the economics of all highway schemes are assessed to justify investment in them. In some locations, should the higher range forecast traffic growth be realized, the design flow may be exceeded within the 15 Year design period. Both these periods are so much shorter than the 100 year design life for bridges that, if we are to make any sense of the later, there is a positive need to ensure that bridge replacement is not initiated by actions emanating from the former. It would be eminently sensible to adopt a policy whereby premature demolition of bridges could be avoided if widening should become necessary in the future. And of course, such a policy should be demonstrably economically sound.

It seems certain that traffic congestion, both on highways and other roads, will continue to be a problem for many years ahead. The demand for increased capacity on the roads is one that the Pakistan government cannot afford to set aside, from whereas there are substantial benefits to be gained in matching it. Nonetheless, the provision of extra capacity is a costly exercise and the search for value for money will always take high priority in delivering an overall programme.

Beyond a certain period of time, it is not economically sensible to make an immediate provision for extra capacity. But, if growth in demand continues, then it may be necessary to re-widen some highways at some time in the future. Experience has shown that the bridges have a major impact on the progress of such work, causing delay and adding to the cost of schemes. It would therefore be prudent, in building bridges with a 100 year design life, to incorporate sufficient flexibility into their design to accommodate some future provision and avoid expensive demolition and replacement of bridges. This is especially true if this can be done at no extra cost to the public purse.

It is important when devising any national programme for the provision of road schemes that there should be an overall balance. Nevertheless, having established that traffic demands are out stripping the existing capacity of the road network, a strong case can be made for increasing capacity along the broad routes followed by the existing highways. Improvement along existing routes supports the creation of new traffic corridors and provides an opportunity to upgrade the environmental measures incorporated into the works when they were built.

Widening of these routes will also enable attention to be more sharply focused on where congestion is greatest and the needs of industry best served and experience has shown that widening schemes on existing roads can be completed in less time than the building of new roads where the time to completion can be substantially extended due to statutory procedures.

It is usually the case, that where a substantial advantage exists a substantial disadvantage will also surface. In the case of widening the disadvantage is the need to maintain the flow of a large traffic volume while the works are undertaken. Many highways are already operating at well above their design year flows ad 30-40,000 vehicles a day is not unusual with some routes as high as 75,000 per day. Imposing restrictions at this level causes frustration adding to the accident risks and inevitably giving rise to considerably increased costs for delays. Thus, there will be a strong impetus towards adopting traffic management measures, which will impose the least possible restrictions on the existing traffic.

Many highways in Pakistan carry larger volumes of traffic than they were originally designed to accommodate. Extra lanes are added as fiscal and other priorities permit in order to ease on increased traffic flow. During the widening construction on bridges, it is usually more economical to maintain traffic flow on an existing bridge with a reduced number of traffic lanes than to construct a detour and reroute traffic. This construction procedure however due to its nearness to traffic on the existing bridge is still very expensive and there is an increase in the public's travel costs due to slower speeds and congestion in the area. Methods to facilitate the widening construction procedure for bridges benefit the department, contractor and the public in terms of direct costs and convenience.

A typical bridge widening consists of the following generalized steps:

1. Placing temporary concrete traffic barriers on the existing bridge next to its exterior girder.
2. Removing the existing curb, sidewalk handrail and portion of deck, leaving required lengths of existing deck bar reinforcement.
3. Constructing a new bridge parallel with the existing, separated by only a narrow closure strip of about two to four feet width, which includes bars from both decks.
4. Restricting traffic to one lane with much reduced speed to prevent severe vibrations for 24 hours when the closure pour is made using high early strength concrete or stopping all traffic on the existing bridge.
5. Removing the temporary barrier after the closure strip concrete is of sufficient strength, and opening the widened bridge to traffic (the effects of traffic induced vibrations on the reinforcement embedded in the closure pours fresh concrete is still not fully understood).

The decision to implement a road-widening programme and to cause the least disruption to traffic means that careful planning and clear policies are paramount. Two principal issues can be identified in this context, which will have a considerable influence on the available options for either modifying or replacing the bridges within the present Programme. They are:

- Bridges are themselves a constraint on the widening methods to be employed.

The choice of replacement bridges will be geared to construction in a live traffic situation and not necessarily to minimum initial capital cost.

Chapter 2

INSPECTION AND LOAD EVALUATION OF CONCRETE BRIDGES

2.1 GENERAL

This chapter is not intended to duplicate detailed handbooks for bridge inspection or the many guidelines published by professional societies and by state and federal agencies. Instead, it offers brief, practical advice, what to look for, how to make inspection findings useful at the probable next stage of rehabilitation, and how to do it safely and effectively. It is meant in equal measure for inspectors in the field and structural engineers faced with rehabilitation design.

Inspectors will find field-tested tips for enhancing safety in the often-dangerous bridge environment, for doing the job thoroughly and efficiently, and for developing the “Practiced eye” that guides experienced field inspectors. Structural engineers will find here an overview of how to put an inspection together, what equipment is available, and what further testing and analysis may be done.

Inspection is the beginning of rehabilitation. Even where the inspector and the structural engineer are not the same person, they often are inspectors in the eyes of the structural engineer. A practical overview of inspection, then, is the appropriate beginning for a book whose primary thrust is the rehabilitation of bridge.

2.2 THE INSPECTION PROCESS

Each inspection is unique. It is therefore not the intention of this chapter to detail how bridge inspections should be performed because inspection process varies with bridge type. The following may, however, provide a common basis to guide inspectors toward an appropriate process for each particular bridge.

2.2.1 Review Available Information

Whether the purpose of the bridge inspection to record the bridge condition, or to develop plans of repair, an inspection always begins with good planning. Review available plans and previous inspection reports in the office. If it is a major or historic structure, it may help to research old engineering magazines and technical journals for articles about the design and construction of the structure.

A minute of preparation can save hours in the field. A thorough review of any existing plans of a structure can identify areas of specific concern and often identify potential problems to the trained eye. This is particularly important when the structure has pins, fracture, critical elements, fatigue-sensitive details, or movable spans. Field sheets and sketches should be prepared checklists and logbooks for recording sketches and photos. Of course, this effort depends on the nature and size of the structure, but no structure should be inspected without some measure of preparation.

If previous inspection reports exist, previously identified facts should be tabulated, with space to note the current existing condition beside each for direct comparison

2.2.2 Coordination

Just prior to the inspection a coordination meeting should be held between all of the inspection team members. No one should arrive at the site without a clear understanding of what is to be accomplished and how to do it safely to the completion of the inspection.

2.2.3 Inform Authorities

Prior to beginning the inspection, it may be necessary to inform the local police if lane closings are required particularly on heavily trafficked highways. In some locations, a permit may be required.

2.2.4 Inspection Notes

From the very beginning care should be taken to write clear and accurate notes, as this will be the record of your inspection. Prepare beforehand a set of bridge plan and sketches of framing plans, deck plan, and pier elevations to locate items you observe and want to report. Review your notes at the end of each day to make sure you did not miss anything.

When taking photographs, keep a record of the roll and negative number of the picture, along with a description, location and direction of picture was taken at. Once back in the office, record the roll and negative number on the back of the photo as soon as possible. Keep the negative separated by roll number so that duplicate photos can easily be made for reports. For long term storage, archival quality (acid free), storage envelops or pages and mounting materials are preferred. (Many ordinary glassine or vinyl products, adhesives such as rubber cement react chemically with negatives and points and can render them unusable in a few years). Try not to use original photos in reports; make a reprint.

2.3 VISUAL INSPECTION OF CONCRETE BRIDGES

The need to detect defects and assess the deterioration level of bridges may be related to safety or to the compilation of an information base for a bridge management system. Precise detection of defects is necessary for bridge repair and rehabilitation.

Concrete is the most widely used materials in bridge substructures and foundations. The main reason being that it is relatively inexpensive and durable. Various techniques have been used to detect defects and structural deterioration, ranging from striking the concrete surface to detect delamination to the radiography of post tensioned members. Some new applications of technology from other fields appear to be promising and examples are infrared thermography and radar.

In the context of this discussion, inspection is the process of assessing the physical condition of a bridge in order to determine corrective action, such as maintenance, repair, rehabilitation, widening strengthening or replacement. There are several inspection manuals outlining the guidelines and articulating the inspection requirements of bridges. Hence, this discussion is confined to the basic objectives of inspection and observations that should be included in any field investigation.

2.3.1 Approaches

Approach roadways should be checked for unevenness, settlement, and roughness. Any of these defects may cause approaching vehicles to induce unnecessary and undesirable levels of impact in the structure. Cracking or unevenness of a rigid approach slab may indicate a void under it from fill settlement erosion.

The joints between the approach pavement and the abutment back wall intended for thermal movement should be checked for clearance. These joints should also be sealed to prevent accumulation of noncompressible materials that may inhibit thermal movement. Shoulders, slopes, drainage, and guardrails should be included in the inspection.

2.3.2 Streambeds and Banks

These are essential features of waterways under bridges. The amount of debris carried during floods should be observed, because inadequate freeboard for ice and debris poses a heat to the structure during high water. Maintaining the channel profile with appropriate revisions as significant changes occur may indicate tendencies towards scour, channel shifting, degradation, or aggradations. As these characteristics unfold, a determination can be made regarding the protection of substructures and foundations. Protective features of existing banks and shores should also be checked to ensure their proper function. Changes in the channel that might have caused the recent protection to be inadequate should be observed and properly assessed.

The inspection should ensure that the waterway is not obstructed but carries the free flow of water. Sand and gravel bars deposited in the channel may redirect flow, causing scour at piers and abutments. In addition to observing the waterway and wave action, the surrounding area should be inspected to determine if the bridge and its approaches are the cause of any problems. These may be possible flooding from inadequate waterway opening, erosion of banks and levees from improper location or skew of substructures.

2.3.3 Piers and Abutments

Footing should be checked for the potential of scour undercutting. This inspection should normally be done at the season of the lowest water, and may include probing or diving. Special attention should be given to foundation on spread footings, because damage associated with scour will be more critical than with pile foundations. However, scour or undercutting or a pier can be quite serious, and hence exposed piles should be properly in whereas the vertical (axial) capacity of piles will normally not be greatly affected, the lateral stability may be markedly influenced (see also subsequent sections).

Exposed concrete should be examined for the presence and severity of cracks and visual signs of deterioration. Tops of piers and abutments are particularly vulnerable to deicing chemical attack, as these materials are carried down by free drainage through open joints and cracks in the superstructure. Structural steel partially encased in substructure concrete should be inspected at exposed faces for deterioration and movement. Stone masonry should be checked for cracks, erosion, cavities, and other signs of deterioration. If any movement or settlement is suspected, piers and abutments should be observed and compared with previous records, because of the obvious structural implications, particularly in continuous spans.

2.3.4 Concrete Girders

T-beams should be checked for possible cracking, especially in the stem area, and for possible concrete deterioration, especially over bearings. Through girders over a traveled way should be inspected for any impact damage from trucks. Girders over underpasses should be inspected for damage resulting from overheight vehicles parking under the bridge. The soffit of the lower slab in box girder structure should be examined for possible cracking, and the same check should be made on the outside face of the girders. Any offset at hinges may indicate bearing problems, and should be identified. An abnormal offset may require further inspection to determine the cause and severity of the condition.

Prestressed concrete girders should be checked for misalignment, cracking, and concrete deterioration. Cracking or spalling may occur in the area around bearings, and at cast in place diaphragms where creep and humping of the girders can produce adverse effects. The nature, size, and location of cracks should be noted and properly recorded.

2.3.4 Bearings

All bearing devices should be examined to ascertain that they function properly. Any Small changes in superstructure and substructure elements may be amplified at the bearings. Bearings devices and shear keys may be subjected to bindings and damage from creep in skewed bridges. Anchor bolts should be checked for possible damage ad secured position. Anchor bolt nuts should be properly set on the expansion bearings to provide the normal movement.

Expansion bearings should also be checked to ensure that they can accommodate free movement, and are free of all foreign material and debris. Rollers ad rockers should bear evenly for the full length and should be correctly positioned relative to the ambient temperature at the time of the inspection. Lubricated bearings should be checked to ensure that they maintain the proper lubrication level.

The inspection should focus on the physical condition of elastic type bearing pads, including any abnormal flattening, bulging, splitting that may indicate overloading or uneven application of loads. Grout pads should be checked for cracks, spalls, and general deterioration.

The concrete at bearing seats should be checked for cracks for crack ad spalls, particularly where girders ad T-beams bear directly on concrete. A check should be made for shearing cracks at the ends of beams and for edge cracks and spalling in supporting members. Bearing assemblies are discussed also in subsequent sections as they relate to common problems ad their causes.

2.3.6 Expansion Joints

Poorly maintained expansion devices should be examined carefully, such devices can be a constant cause of problems. The inspection should focus on the joint adequacy to accommodate the thermal movement, the joint opening under full contraction, and the tendency to collect and accumulate debris. Sealed joints should be checked to ensure that the seal prevents entry of noncompressisble items and debris. Finger type joints and sliding plates should be examined for evidence of loose anchorages, weld cracking or breaking, and other defective details Besides causing structural damage, defective joints may also be a traffic hazard. The concrete adjacent to expansion devices should be sounded to detect voids or laminations in the deck. The underside of expansion joints should be examined for possible or impending problems. Where the joint opening is not enough for expansion, thermal movement induces shear stresses and causes the concrete to shear and spall

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2.3.7 Concrete Decks

Concrete decks should be checked for cracks, leaching, scaling, pot holing, spalling, and other signs of deterioration. Signs of deterioration in the reinforcing steel must be examined closely and properly analyzed to determine the type and extent. Decks treated with deicing agents or locate in "Salt" environments should be carefully checked because of their vulnerability to chemical attack. Asphaltic or other types of wearing surface on a bridge deck usually tend to hide the defects until they are well advanced. These surfaces must be carefully examined and any signs of deterioration properly noted and recorded. Visual defects may include cracking or breaking up of the surfacing and excessive deflection. Where deterioration is suspected, the surfacing may be spot checked by removing small sections for observation and inspection. The underside of deck slabs should also be examined for any signs of deterioration distress, such as water passing through cracks. Where permanent deck forms have been used, panels should be removed as needed for inspection.

2.3.8 Curbs, Sidewalks, and Railings

Concrete curbs must be examined for cracks, spalls, and general deterioration. Any loss of height because of buildup surfacing on the deck should be noted and recorded. The condition of paint used for improved visibility should also be checked. Likewise, concrete sidewalks should be examined for cracks, scaling, spalling pot holing, and other forms of deterioration. The condition of joints especially at abutments should be noted along with any differential movement that could open the joint or make an offset hazardous to pedestrians. Sidewalks should also be inspected for proper drainage and surface roughness. An excessively rough surface may be a hazard to pedestrian traffic and should be properly noted.

Concrete handrails should be examined for cracks, spalls, scaling, and damage from impact. Handrails should be checked for possible damage from traffic impact. Interestingly, substructure settlement or deficiencies in the function of bearings may show in the railings as deviation in the vertical and horizontal alignment, Joints in railings should be checked for proper function. Handrails should be secure and relatively free of slivers or projections that may create hazards to pedestrians.

2.3.9 Overall Assessment

Any defects and problems determined from the initial inspection will most likely require further investigation to determine the exact nature, extent, and probable cause. Some defects may have a simple readily evident cause. Others may require considerable time and testing to assess and evaluate. The inspection team should not underestimate the significance and usefulness of visual inspection. Bridges should be observed during the passage of heavy vehicles to determine effects such as excessive vibrations and deflection. If neither is detected, further investigation will be necessary until the cause is established. Some measurements may in this case be necessary and should include line, grades, and relevant dimensions.

A visual inspection can identify possible fire hazards, such as accumulation of debris, drift, weeds, and brush, and flammable materials stored under or near the structure. Underwater inspection combines sounding to locate channel bottom, proving to determine deterioration and losses, and diving to inspect and measure bridge components. The assessment of deficiencies of substructures below the waterline is discussed in detail in the following sections, and includes a complete review of improved methodologies and detection of underwater problems

2.4 NONVISUAL INSPECTION

It appears from the foregoing sections that the need to assess defects and the physical conditions of bridges is related to safety requirements and to the load capacity of a structural system. Ultimately this assessment provides the database of a bridge management system. Besides the visual investigation summarized in section above, recent developments in refined nondestructive testing (NDT) techniques have advanced the current inspection level, and enabled technical evaluations that have enhanced decisions on bridge improvement options.

The advantage and disadvantage of nondestructive test procedures for field and laboratory use may be combined with those of destructive techniques that can be performed in the laboratory on samples removed from the structure. However, the emphasis appears to be on test procedures that can be used in the field and on criteria to be used in interpreting the significance of obtained results.

Areas that are not accessible to the naked eye may be inspected by drilling holes. This method is most common in concrete structures and especially in pretensioned box beams and voided post-tensioned thick slab decks, usually built without inspection hatches. A relatively small hole (1 in or mm diameter) permits examination of the interior of closed voids, using a tubular instrument (bore scope). A major disadvantage of the bore scope is that the angle and the field of view are shallow and limit visibility.

Larger holes, made with a core drill, enable visual observations with the help of an electric light source and a mirror or periscope. This technique has been used to examine inaccessible areas between the ends of beams and the ballast wall of abutments. Video cameras may supplement the work, although it is difficult to find the proper equipment suited to this task. The defects and deterioration to be recorded are essentially the same as in ordinary visual inspections, and include cracks, scaling, spalling, and areas of honeycomb.

2.5 FIELD TESTING TECHNIQUE

The testing techniques reviewed at the beginning of this section have been refined to produce reliable and sophisticated methods for field-testing. Details as a complete review are provided by the references listed in the text. Hence, this discussion is limited to a brief summary.

2.5.1 Rebound and Penetration Methods

These methods are used to measure hardness. Although this property is not very important in concrete, it may be used as a means of predicting the strength of concrete. Most of the surface hardness tests involve indenting the concrete surface and measuring the side of indentation as in metals. An alternate method is the rebound method, which uses a special hammer (sometimes called the Swiss hammer). It has been suggested that the accuracy can be improved if the rebound hammer is used in conjunction with ultrasonic testing.

An extension of the rebound method is to use a more powerful impact device that can penetrate the concrete. The best known example is the Windsor Probe, which penetrates upto 2 inch(50 mm) into the concrete so that the results are less influenced by surface moisture, texture, and carbonation. However, the size and distribution of the coarse aggregates in the concrete affect the probe results. In addition, the technique is not completely nondestructive, because it sometimes shatters the concrete surface and the probes must be removed and the surface patched.

Nevertheless, the instrument has been found useful in examining concrete components such as bridge piers, where coring would have been difficult.

2.5.2 Stress Wave Methods

The theoretical principles of these methods were briefly presented in the foregoing sections. The associated relationships were derived assuming the medium to be homogenous, isotropic, and perfectly elastic. Heterogeneous materials may be included, provided that the dimensions of the specimen are larger as compared to the constituent materials.

The basis of the dynamic test methods for concrete is that the wave velocity is related to the elastic modulus,it is a function of the void content and hence is related to the concrete strength (Jones, 1970). Thus, there is some basis for empirical relationships between wave velocity and strength.

Pulse-velocity methods are conveniently grouped into two categories.

(1) **Mechanical sonic pulse-velocity methods**. These involved measurement of the time of travel of longitudinal waves generated by a single impact hammer blow or by repetitive blows.

(2) **Ultrasonic pulse-velocity methods**. These involved measurement of the time of travel of electronically generated mechanical pulses through concrete, the time interval being measured by a digital meter and /or a cathode-ray oscilloscope.

2.5.3 Magnetic Methods

The main application of magnetic methods in the testing of concrete structures is in determining the position of reinforcement. Although not strictly a technique for detecting defects or deterioration, inadequate cover may indicate corrosion-induced deterioration; hence, locating the reinforcement may be an important phase of a condition survey. Magnetic methods have also been applied in detecting loss of section or fracture of prestressing steel, mainly in beams.

2.5.4 Electrical Methods

Electrical methods used in the field are generally limited to resistance and potential measurement. Examples are measurements for determining moisture content and polarization studies for measuring corrosion rates. However, equipment suitable for field use in the developmental stages, and improvements should be forthcoming.

Resistivity is one of the factors that control the rate of corrosion of steel in concrete. Resistivity is normally measured by the four-electrode method common in geophysical testing. Four contact points are placed in the concrete equally spaced in a straight line. An alternating current is passed between the outer electrodes. The resulting potential difference between the inner electrodes is measured and the Resistivity is estimated from the expression.

$$P = 2na\text{E/I}$$

Where, p = resistivity

a = electrodes spacing

E= potential drop between P1 and P2

I = current flow between C1 and C2

Corrosion may occur in the following Resistivity ranges:-

a. If Resistivity exceeds 12, 000 Ohm-cm, corrosion is unlikely
b. If Resistivity is in the range 5,000 to 12, 000 ohm-cm, corrosion is probable if Resistivity is less than 5,000 Ohm-cm, corrosion is almost certain.

When steel corrodes in concrete, a potential difference exists between the anodic half-cell areas and the catholic half cell areas along the reinforcing bar. The potential of the corrosion half cells can be measured by comparison with a standard reference cell, which has a known constant value. The call cell can be used vertically downwards, horizontally, or vertically upwards, provided that, a copper sulfate solution is in contact with a porous plug and the copper rod in the cell at all times.

On bridge decks that have a seal coat or membrane, the seal or membrane must be punctured at the point of measurement. A full description of the equipment and the test procedure is given by ASTM C876.

2.5.5 Chemical Methods

General chemical test methods applicable to concrete structures usually involve determination of the depth of carbonation and chloride ion content. Both methods are used to establish the possibility that the passivity of the reinforcement might have been destroyed.

A method has been developed for measuring the chloride content of bridge decks in situ. The procedure involves drilling a hole with ¾-in diameter (19 mm) in the deck to a predetermined depth. The hole is then filled with a borate-nitrate solution, and a chloride-specific is inserted. After 90 seconds, the potential across the electrode is measured and the chloride ion content is determined from a calibration curve. The method is speedy and causes minimum damage to the deck, however, it requires a high level of expertise, which may be a deterrent to its use.

2.5.6 Nuclear Methods

The main application of nuclear methods is, in measuring moisture content in situ. Laboratory testing has demonstrated, that this approach may be feasible in the field, in providing reliable moisture content data as a function of depth. However the associated equipment is reported to be expensive, heavy, and requiring skilled operators.

2.5.7 Thermography

Infrared thermography has been found feasible in detecting delamination of bridge decks. It can also be used on other concrete elements, such as columns, provided that they are exposed directly to the sun. The underlying principle is that as the concrete heats and cools, there is a substantial thermal gradient within its volume, because concrete is a poor conductor of heat. Any discontinuity, such as a delamination, parallel to the surface interrupts the heat at transfer through the concrete. Thus in periods of heating, the surface temperature of delamination is higher than the surrounding concrete. The difference in surface temperatures can be measured using sensitive infrared detection systems.

2.5.8 Radar

The use of ground-penetrating radar systems for detecting deficiencies in bridge decks began in the mid 1970s. This potential was stimulated by the development of low power, high frequency pulsed radar in the 1960s. A number of studies have been carried out on both bare concrete and asphalt-covered bridge decks. In all cases, the radar was found to be capable of identifying anomalies in the deck. A practical problem was analyzing the large amount of data that were collected and relating the radar output to physical distress and specific deficiencies. An important advantage of radar is the ability to measure the thickness of the asphalt surfacing.

2.5.9 Radiography

The principle involved in this method is that as the radiation passes through a material of variable density, more radiation is absorbed by the denser parts.

2.5.10 Air Permeability

Air permeability measurements may be applied to obtain an in-situ assessment of the resistance of concrete to carbonation and penetration of aggressive ions. It consists of drilling a hole into the concrete. The air in the hole is evacuated and the rate at which this occurs is correlated with the permeability of the concrete.

2.5.11 Comparison of Test Methods

Some of the techniques for detecting corrosion of embedded steel measure corrosion activity and level directly, whereas others measure concrete properties that relate to corrosion. A classification of the techniques that may be used in examining reinforced concrete structures for steel corrosion be separated.

A good visual survey, supplemented by a proper selection of testing methods, is a sound approach. Detecting corrosion of embedded reinforcement would normally require a proper technique such as concrete cover, half-cell potential, and chloride-ion content. Taking cores for laboratory examination may best assess the nature of chemical attack and aspects in the quality of concrete that contribute to scaling, wear, or abrasion damage may be best assessed by taking cores for laboratory examination.

2.6 LABORATORY ANALYSIS

Method of laboratory test methods (ASTM and AASHTO methods) for concrete is standardized and relatively straight forward. A brief description is presented in this section.

2.6.1 Petrographic Analysis

This provides a detailed examination of the material composition, and structure of concrete. Items that can be identified include the mineral aggregates, the paste-aggregate interface, and the structure and integrity of the paste. A petrographic analysis can be extremely useful in articulating mechanisms of deterioration such as freeze-thaw, sulfate attack, and alkali-aggregate reactivity. The procedure can also be used to assess the severity of fire damage.

2.6.2 Physical Test

The strength of concrete can be measured directly in compressive tests of core samples. The modulus of elasticity can be determined from the linear portion of the stress-strain curve. The tensile strength can be measured on core samples by the indirect tension or Brazilian test. These tests involve standard ASTM and AASHTO procedure. However, core samples for strength tests should be free from reinforcement, a condition not always possible.

Horizontal bars through the core tend to reduce the measured strength because they act as stress concentrations, but there is no agreement as to the correction factor that should be applied for a given configuration of reinforcement to give the equivalent strength of plain concrete. Cracked cores should not be tested for strength.

The density of absorption of concrete samples can be measured in simple tests. A measurement of the air-void system of the concrete by either the linear-transverse or modified-point-count method can be used to determine its resistance to freezing and thawing.

A relatively new method is the rapid measurement of chloride permeability. This test consists of monitoring the amount of electrical current passed through a 2-in long core with one end immersed in sodium chloride solution and a potential difference of 60 V be maintained across the specimen for 6 hours. The technique is best suited to the evaluation of sealants and modifiers to decrease the permeability of concrete, but can also be used in assessing the overall concrete quality and resistance to penetration by chloride ions.

2.6.3 Chemical Tests

These are important for carbonation and chloride-ion content. The methods can also be used to test the cement content, although the results are not always reliable. The depth of carbonation is measured by indicator solution using the same methods as in field-testing. The chloride-ion content is measured by a wet chemical process in two ways, depending on the method of extraction. The total chloride-ion content is measured by dissolving a powered sample of the concrete dilute of Hardened Concrete.

2.7 UNDERWATER INSPECTION

The objectives of underwater inspection can be defined as the combined effort to locate the channel bottom, probe to determine deterioration and losses, and dive to visually inspect and measure bridge components.

Because underwater inspection entails conditions that require experience and familiarity with sub aqueous environments, underwater activities are usually left to those who have proper training. Even in shallow depths, the physical effects of this work cannot be predicted without a through physical screening. Thus, physical and mental capabilities of the underwater inspector must be sharp with emphasis on safety and safety procedure to ensure comfort and techniques success while working underwater.

A great deal can be accomplished with the use of underwater video and photography to inform the engineer of conditions below. Video can provide views of all accessible angles and sections, while the video images are monitored from above and communications are maintained. Voice contact is mandatory and can be provided with a conventional communication system.

As discussed earlier visual inspection is invariably the first test in underwater inspection. Visual assessment is quick, practical, and nondestructive. However, its success may be limited because of the water turbidity and the poor visibility. In addition, the method gives qualitative results, and cannot provide a direct measure of strength.

2.7.1 Physical Measurement

Physical dimension measurements provide data on section loss and reinforcing steel. The method is relatively quick and easy, and the results are quantified. Because a minimal amount of time and equipment is involved, the cost is nominal. The procedure is nondestructive and applicable to underwater use. Its main drawback is that the information is partial and does not extend to the remaining concrete.

2.7.2 Soundings

Soundings are taken by striking the concrete surface to locate areas of delamination of the concrete cover caused by cycles of freezing and thawing and corrosion of the reinforcement. The method is economical but not particularly effective, because the inspector's ability to hear and distinguish sound is affected by waves, current, and background noises

2.8 TESTS FOR UNDERWATER INSPECTION

Testing techniques must be assessed for ease and speed of application, destructiveness, underwater usefulness, provision of quantitative strength measurement, and cost effectiveness.

2.8.1 Cores

Core samples, taken from underwater concrete, may be used for petrographic analysis and other laboratory procedures measuring compressive strength, diffusion constant, permeability, electrical Resistivity, density, moisture content, chloride penetration, carbonation, and air entrainment. The method is relatively destructive, in that, it causes micro cracks and eaves holes. Because coring is moderately expensive, cores are usually taken where other evidence indicates that further investigation is warranted.

2.8.2 Ultrasonic Pulse Velocity

This technique under stress-wave methods, allows measurements to be taken underwater at the time of transmission of an ultrasonic pulse of energy through a known distance of concrete. The results can be affected by several factors, including aggregates and steel bar location, and must be correlated with other tests such as coring to obtain absolute value.

2.8.3 Voltage Potential Readings

These are taken to assess the state of corrosion of the steel bars by making an electrical, connection with the reinforcing steel. The most satisfactory results are obtained when the readings are used in connection with a chloride penetration analysis of a core test. The reading indicate whether corrosion is active and the area of concrete involved, but they do not give the rate of the amount to corrosion.

2.8.4 Polarization Resistance Measurements

These techniques can actually measure the rate of corrosion. Several techniques are available to apply this technology, such as the AC impedance, and the two-electrode and three-electrode methods. The results may be analyzed according to various procedures, including the slope of the polarization curve near zero current method. The polarization resistance approach is still being developed for underwater testing, and should be considered in the light of the most recent advances.

2.8.5 Computer Assisted Tomography

Computer-assisted tomogrophy (CAT) is the group of nuclear methods, and uses a nuclear source, to develop a cross-section view of a member. It is nondestructive and can scan members up to 3 ft thick. The method is expensive, and its underwater application is still experimental and under development.

2.8.6 Other Methods

Other methods that may be developed for underwater use are pullout tests, penetration tests, indentation tests, and resonance tests.

2.9 FUTURE TRENDS IN INSPECTION

As the art of inspection becomes more developed, and the specialized needs in the inspection of bridge structures are recognized, more advanced equipment and testing methods are being researched and developed. Some have already been mentioned. Those noted below are in some cases already proving their effectiveness in the field, whereas others are still being developed. All, however, are worth bearing in mind as possible options, particularly for situations that challenge conventional methods. In all cases, literature from manufactures be obtained, on the latest models available, and on the question of exactly what situations each sort of equipment handles best at this stage of its development.

2.9.1 Pulse-Echo

One recent advancement, specifically for concrete inspection is the pulse-echo machine. Similar to the ultrasonic gauges used for steel structures, the device uses acoustic cracks and flaws in concrete elements.

2.9.2 Fiber scope

The fiber scope is also becoming useful tool in inspection. Able to be inserted in small, formerly inaccessible areas, such as the tendon duct for a prestressed cable, the scope carries a tiny (quarter-inch) television camera at the tip of a fiber optic cable. The camera’s image is transmitted back to the monitor of a videocassette recorder (VCR) and can be recorded for later playback. Current technology limits color and cable articulations control to 20 feet, with black and white imaging available with cables upto 100 feet in length.

2.9.3 Fiber-Optic Sensors

The National Foundation has sponsored research on embedding fiber optic sensors into bridge decks or mounting them on structures to monitor any structural deformation or change of alignment. Light pulses sent from one end of an element to another will be received with slight changes of position or timing as elements deform or shift, thus giving an early warning of conditions that may need inspection and rehabilitation.

2.9.4 Magnetic Perturbation and X-Ray

Nondestructive testing instruments are being developed for the specialized inspection of suspension bridge cables and cable-stayed bridge stays. Currently, the cable has to be unwrapped and spread open with oak or plastic wedges to check the condition of individual cable wires, a time-consuming and costly procedure. One newer instrument is the Magnetic Perturbation for Cables (MPC) system, an instrument developed by the FHWA and was first time demonstrated in field at the end of 1988.

This self-propelled device can be installed directly over the wrapped cable in the field or laboratory, and consists essentially of an electromagnet with an attached magnetic flow sensor. It locates and records flaws or breaks in the individual strand wires by detecting flux leakage as the device travels along the cable bands. Certain features of construction, such as grout integrity in cable-stayed bridge stays, can also be verified, all without unwrapping the cable. Another developing inspection tool is the X-ray inspection of cable sockets, using a miniature electron linear accelerator.

2.10 Structural Assessment Considerations

A usual procedure for making a safe load determination involves a logical sequence of calculations for the deck floor (usually a concrete slab), stringers, floor beams, and girders or trusses.

The floor is seldom the controlling element in a bridge with longitudinal beams. However, when computing stresses in the concrete or reinforcement, the live load distribution should be according to the AASHTO standard specifications (1992 edition). Moments in beams and girders may be computed analytically or determined with the help of design aids, charts, and tables. Design aids are also available for live load moments for the three typical AASHTO loads. The same procedure may be used to determine live load moments in floor beams.

Likewise, live loads in truss members may be calculated by using appropriate formulas for maximum shear and moment given by design aids in the AASHTO specifications unless a more analytical approach is used. Reference to these formulas will give the maximum live load stress for the three typical truckloads. These formulas are valid only where used within the given limits. Modifications will be necessary when the live load does not meet these limits, for example, when the structure or panels are too short to permit the entire load to be placed and positioned to produce maximum shear or moment.

2.10.1 Allowable Stress And Load Factor Methods

Highway bridges are usually rated in accordance with the current edition of the AASHTO Manual for Maintenance Inspection of Bridges. Bridges can be rated by the allowable stress method, or load factor, as approved by the owner. For some older bridges when the proportion of dead load is high, such as for concrete-encased steel beams, load factor may be a more appropriate method of rating.

For railway bridges, the AREA Manual, as modified by the bridge owner, applies. For transit systems, either the AASHTO or AREA standards may be used.

Individual members should first be rated for their full design section based on today's rating criteria. This will determine the maximum capacity of the bridge if it is rehabilitated to the original design condition. Members should then be rated for current loss to determine their present load-carrying capacity.

When using allowable stress design, the allowable (working) stress should be taken from the AASHTO specification, contained in the Manual for condition Evaluation. However, there may be cases that warrant a reduction in the allowable stress, based on a judgment of the quality of the material under consideration. This determination will apply more often to timber members, because of the effects of deterioration and weathering on strength. Relevant data used as basis for selecting allowable stresses, should be noted at the time of the field investigation.

Conversely, a higher factor of safety for a bridge, carrying a large traffic, may be desirable when compared with the same criteria applied to a bridge that carries few vehicles, especially if the former includes a high percentage of heavy loads.

Alternatively, AASHTO permits the rating of bridges by the Load Factor Method, taken from the standard specifications (1992 edition). This approach may be used as an alternate to the ASD.

The working stress method simply means that the ultimate limit state is automatically satisfied, if allowable stresses are not exceeded but, depending on the variability of materials, this is not always true. Thus, it is often necessary to consider the deflection limit state, and for concrete the crack-width state. Inconsistencies have been pointed out, the most serious shortcoming being the inability to quantify the variability of resistances and loads, the approximation of the level of safety, and the inability to consider groups of loads where one increases differently from the others.

The load factor method (also referred to as strength design method) is essentially limit states design with emphasis on ultimate limit states, with the serviceability limit states checked after the original design is completed. According to this philosophy, the required strength of that must be developed to resist the factored loads and forces applied to the structure in combination stipulated in relevant criteria. The design strength is equal to the factored resistance and the required strength is the sum of the load effects computed from the factored loads $Y(\sim IP+(L+I)+\ldots)$.

According to the load factor design, engineers may choose to proportion simple and continuous beam and girder bridges by considering multiples of the design loads. Furthermore, to ensure serviceability and durability, the design must also address the control of permanent deformations under overloads, fatigue characteristics under service loads, and control of live load deflections under working conditions.

The extension of the load factor approach to foundation systems and to the soil-structure interaction is one of the main features of the new AASHTO-LRFD specifications. A key point in the design of substructures and foundations is that uncertainties may unfold from four main sources.

(1) Uncertainties in estimating loads and their distribution;

(2) Uncertainties associated with the variability of soil conditions at each sub-structure location

(3) Uncertainties in assessing in situ engineering properties of soils and rocks, and

(4) Uncertainties with regard to the degree to which the analytical model can predict the actual behavior of the soil-structure system. For older structures subjected to deterioration, uncertainties may also relate to material properties and structural strength available in the system components.

Structure as a whole and their components should be proportioned to resist sliding, overturning, uplift, and buckling. The later is particularly critical for compression members subjected to lateral eccentricity of loads.

In addition, structure should be proportioned to resist collapse due to extreme events.

2.10.2 Computing Load Capacity

To determine the load capacity of a highway bridge, the allowable capacity of each member is first computed for the rating level(s). This allowable capacity is then reduced by the actual dead load carried by the member to produce the capacity for live load. Dead load can usually be computed from the original design plans and modified to take into consideration any overlays on the bridge deck or other changes in the structure since the original construction. (It is important that the changes be documented during inspection). The live-load capacities of the members are then compared to the live load calculated in the member by placing various truck types within the lanes on the bridges. The live-load capacity can then be determined for each member, and then the bridge as a whole, in terms of tons for each of the trucks rated. AASHTO calls for a rating in terms of H20 and HS20 design truck types 3-1 (regular two-axle truck), type 3-2 (a trailer truck, eighteen wheeler), and type 3-3 (tandem truck). Some states require using an HS25 truck in recognition of the trend for heavy loads on today's roads, or for other specific truck configurations allowed within their borders.

Railroad bridges are usually rated in terms of" Cooper E loadings. A Cooper E loading of 72 or 80 is commonly used for design today. Older railroad bridges were usually designed for a Cooper E-60 loading or lower. However, heavy impacts from steam locomotives were included at that time.

Impact is not as great as with today's diesel and electric engines, which tends to mitigate the lower original design loading. The rating is also mitigated by the fact one calculates the load based on the specific equipment in use on that line in order to more accurately evaluate the bridge capacity.

When considering the load capacity of a structure one needs to also consider its fatigue life, and this is discussed later.

2.10.3 Load Testing

When the capacity of a bridge must be more accurately known, it may be necessary to load test the structure. Strain gauges are placed on critical members of the unloaded bridge. Then, the structure is control loaded, using trucks, transit cars, or railroad engines of known weight placed at various points across the structure, and the additional strain is recorded. In this way, the actual load distribution to the various members can be measured at this time using survey instruments. Experience has shown that there may be significant differences: "as tested" a bridge may have greater capacity than "as computed," because of the actual distribution of the load to the members.

A recent development in field testing is "weigh-in-motion" studies. Strain gauges are placed on the structure, and the stress is recorded as vehicles pass over. This is useful for determining the reaction of the structure to normal traffic loads and, by continuous monitoring, it can also be used to determine the loads crossing the bridge.

Chapter 3

INSPECTION AND LOAD EVALUATION OF ARCH BRIDGES

3.1 ARCH BRIDGES

As a structural unit, an arch is defined as a structure shaped and supported in such a manner that intermediate vertical loads are transmitted to the supports primarily by axial compressive thrusts in the arch. In addition, the structure must be sustained by supports that can develop lateral as well as normal reaction components. For a given loading the arch shape must be chosen so as to avoid the introduction of bending moments. For typical downward loads this shape will concave downwards, and in this case the arch becomes the exact inverse of a suspension bridge cable.

3.2 DESCRIPTION OF MODERN ARCH BRIDGES

Arches during the Roman and Renaissance periods were characterized by stone masonry configurations, described ‘as the voussoir or arch block type and clearly distinguished from modern monolithic elastic structures. The later are analyzed using elastic theory whereby the entire structure is assumed to be a monolithic elastic unit where the true crown thrust is determined using equations taking into consideration the deflection or elastic distortion of the materials under load.

A review of arch bridges must address the following items.

1. Materials of construction
2. Structural arrangement of the arch
3. Stress distribution in the arch
4. Methods of supporting superstructures.
5. Shapes or curve of the arch.

The factors, individually or combined, determine arch performance, hence they influence the development of problems and deficiencies as the structure begins its service life.

3.3 INTRODUCTION

Most of the Arch structures are over 100 years old, the design life for modern structures, but are required to carry loads much greater than their designers could have envisaged. The ever-increasing weight of vehicles, especially road vehicles, means that many of these structures must be strengthened.

The masonry arch is a very durable structural form but it is difficult to analyze. Consequently, designers of masonry arches used rules of thumb to proportion their structures. Recently, sophisticated arch analysis techniques have become available, for example the SAP 2000 non-linear computer package, and this has enabled the true capacity of arches to be determined. Frequently the arch has sufficient strength to carry modern loadings. All that is required is to ensure that the structure be in good condition. Thus the repair of an arch can amount to strengthening.

Arch defects can develop for many different reasons including weathering, ground movements, and breakdown of waterproofing systems or the effects. Strengthening may be required due to these defects or because of an increase in loadings. In either case, an assessment of the structure will be required and the engineer must decide whether to repair the structure. There are two basic reasons that will lead to repairs being carried out. The first is if a fault is assessed as directly affecting the arch in terms of limiting the live load. The second reason is that although the fault may not currently restrict the live load capacity of the bridge. Failure to repair the fault may lead to further serious damage developing. The correct identification of the reasons for repairing the structure is important because it governs the criteria by which the success of the repair should be measured.

In this chapter, the subject of arch condition assessment will be briefly discussed as it is the process by which the capacity of an arch is calculated and therefore the process that limits traffic over the structure.

3.4 IN DEPTH INSPECTION

The first phase includes in depth inspection of the basic structure, including pavement, curb, sidewalk, spandrel walls, parapets, concrete arches piers abutments, fender systems waterway and marine traffic approaches sign structures and utilities, Various inspection equipment and techniques be used to prepare the engineering report, including an innovative use of photographic techniques. Following the inspection a detailed evaluation of the structural behavior and capacity of the existing concrete arches reveals what and where repairs will be necessary. Concrete core specimens should be obtained and be subjected to petrographic analysis, compressive strength tests, air content tests, freeze-thaw tests, and chloride tests.

3.5 INSPECTION PROCEDURE

The inspection is performed by a team. Inspection equipment is used for close-up inspection of the underside of the arches, above water areas of piers, and timber fender systems. Safety and economy must be ensured. Photography is used to prepare a reconstruction project report containing results of in-depth inspection, stress analysis, photographs. Representative of each type of problem encountered, recommendations, and construction cost estimates of rehabilitation work proposed. This report would be reviewed by the client to help establish the definition and scope of services for the final design document phase preparation of contract drawings, specification, and detail estimate.

Rehabilitation/widening projects dictate that a significant amount of time be spent in the field to properly evaluate the present condition of all structural elements. This in turn will allow preparation of alternate design solutions for rehabilitation. A photograph has some unique qualities. It can be very selective and can emphasize selected areas in great detail quickly and accurately. In a day or two almost every square inch of a structure can be permanently recorded. With tools such as telephoto lenses and underwater cameras inaccessible areas can be inspected and recorded. But perhaps the biggest asset of a photograph is that it can be easily and instantly understood by anyone, without long explanations.

3.5.1 Concrete Arches

The underside of the concrete arches be sounded throughout with chipping hammers areas containing foreign matter, honeycombing wetness, or cracks be examined extensively. An attempt be made to determine the depth of the faults by using cold chisels, awls, wrecking bars, and pneumatic powered chipping gun. Cracks with efflorescence and construction joints may also be chipped out to determine the extend of the cracking or deterioration.

3.5.2 Underwater Inspection

Underwater inspection of all pier surfaces and timber fender systems be performed by experienced professional divers specialized in underwater inspection of bridges or similar structure.

3.5.3 Core Program

Supplementing the visual and hands-on-inspection procedures and throughout the structure, a drilling and coring program with appropriate follow up testing be undertaken. Vertical drilling is done in the approached payments at different locations. The test program includes petrographic examinations, compressive strength tests, air content tests, freeze-thaw tests and chloride tests. Causes of deterioration should be evaluated for two purposes: to find out if there is a way to rehabilitate and upgrade the bridge and to evaluate the effect of deterioration on the strength of the bridge.

3.6 STRESS ANALYSIS

3.6.1 Existing Construction

The roadway over the arch spans is supported directly or through some medium like compacted earth fill. Live loads are distributed to the arch members and then to the foundation. A typical lane width should be investigate individually for dead, live, and temperature loadings. Separate analysis is made for each of the six AASHTO live loadings and for the original design live load. The resulting stresses be then combined in AASHTO loading Groups I and IV together with appropriate live load impact factors to determine the maximum stresses in the concrete arches. A computer model with numerous discrete elements be established for the purpose of finite element analysis of the typical arch structure.

3.6.2 Results of Analysis

An envelope of maximum stress is determined for each of the arches based on both fixed and hinged end conditions. These may then be combined to produce an envelope of maximum stress. Under the assumption of full end fixity and a stress factor of 1.25, the values for required concrete strength to be found in compression and in tension. Based on this reasoning concrete strength be concluded that would enable the arches to adequately support the required loads.

3.6.3 Concrete Quality

To determine the quality of the concrete in the structure, a number of tests should be perform on specimens taken by core drilling. These tests can be the compressive strength of concrete petrographic analysis, sodium chloride content, percentage loss of standard freeze-thaw tests, and air entrainment. A no of core sample be taken from various locations in the different concrete arch spans. The conclusion to be drawn from the results of the concrete quality tests, the concrete considered is acceptable, and this will allow a final judgment that is it feasible to repair and upgrade the concrete arch structures.

3.7 STRENGTH EVALUATION

3.7.1 General

In order to assess the load carrying capacity of the concrete arch spans, a determination of the concrete compressive strength is required. To provide the data for this determination, specimens can be taken from the hardened concrete obtained by core drilling and then compression tests be made on samples prepare from these tests specimens. The design concrete strength be determined by use of the procedures provided in the ACI standard “Recommended Practice for Evaluation of Compression Test Results of Field Concrete.”

Since all present day design specifications are based on concrete compressive strength at the age of 28 days, although it is recognized that all concrete gains strength with age during the first few years after construction. The magnitude of this strength increase is dependent on numerous factors such as type of cement, curing, temperatures, and water-cement ratio, and can vary from 35 to 100 percent of fc at 28 days. Unfortunately, there are no modified design specifications available at present for rehabilitative design. Therefore, to properly compare the required strength of the concrete from the results of stress analysis with the actual strength of the concrete available in the field, and to provide a consistence factor of safety in new design and rehabilitative design, it is necessary to concrete strength to 28 days strength.

3.7.2 Measured Strength

Number of compression tests be made on specimens taken from the concrete arches. These test resulting be converted to the standard height-to-diameter ratio and determine the average design strength for the old concrete.

3.7.3 Corrected Strength

This value of design strength for old concrete had to be corrected to a 28-day value. Usual procedures for relating concrete strengths at different ages involves use of maturity computations for a various ages. Standard maturity computations are invalid, however, for concrete that has long since been completely hydrated. Therefore, in the absence of any valid procedure, a conservative estimate of 75 percent gain in strength can be made. The value for 28 day design strength should then be calculated.

3.8 ALLOWABLE STRESS

While the actual concrete strength in the structure had been established and also the design strength has been established, an acceptable stress level still needs to be determined. In the early 1900s the allowable compressive stress permitted was 0.18 fd. The coefficient 0.18 anticipated the probable variable quality and therefore strength of the concrete. However, in determining a design strength if, this variability of concrete strength has already been considered than higher coefficient, 0.40 (permitted by AASHTO) be used.

Under the discussion of concrete quality, it was noted that the concrete strength test results should be modified by a factor of 0.75 to represent all of the concrete in the structure. This 0.75 multiplier be applied to the computed allowable stress value and a net allowable working stress be recommended as a reasonable limiting value for the existing concrete in the arches. Alternatively, it could be argued that it was not necessary to correct the concrete strength to a 28-day age and the actual strength should be used as fc. The allowable stress, using this approach, should be computed by using the coefficient of 2.18 without any further modification for concrete quality.

3.9 CONCLUSIONS

Therefore if stress calculated is a reasonable working stress for the existing concrete and from the stress analysis we know that if the required concrete strength in the arches is less, we can conclude that the arches, when rehabilitated, will adequately support the loads.

Other general conclusions can be:

- Rehabilitation of the bridge is feasible and practical.
- All bridges elements can readily be upgraded to support all of the AASHTO live loadings.
- The basic concrete arches are reasonably sound and after removal of roadway payment, fill, and spandrel walls, can be rehabilitated and capped with a continuous strip of reinforced

concrete. Other rehabilitation measured would include introduction waterproofing membrane in conjunction with new spandrel wall construction; installation of replacement fill, road, and sidewalk pavement; and total replacement of all drainage systems.

- Pier, and abutment structures are adequate and can simply be rehabilitate and continued in service.
- Modifications could and should be made to existing utilities lighting, communication, and water, sanitary – to bring them up to current standards and operating efficiencies.
- Introduction or upgrading of a number of features relating to pedestrian, vehicular, operational, and mime safety to insure compliance with modern standards are required.

3.10 ARCH ASSESSMENT

Arch assessment consists of calculation of the load capacity of the arch barrel and also the formation of a subjective opinion of the stability of the arch's spandrel walls.

The most commonly used method of arch assessment in the modified MEXE method described in the Department of Transport advice note BAIG/93 (ref. 2). With Modified MEXE Method assessments the capacity of a parabolic arch in perfect condition in initially calculated. This is modified by the applications of factors to account for differences in geometry, materials and faults between the real arch and the ideal structure on which the calculation was based. The method is quick and simple to apply but suffers from several disadvantages.

- The safety margin (the relationship between the permissible load and actual arch collapse load) is nuclear.
- The method is inflexible, preventing modification by the assessment engineer to account for different situations, for example arches with hunching.
- The use of crude factors to take account of arch geometry, materials and faults is obviously not entirely satisfactory.

Various computer programs have been developed to improve the assessment of arch barrels. These programs are not as quick to use as the MEXE method as they require more information about the structure then MEXE does. Nevertheless, an initial assessment can be performing quite quickly using conservative assumptions of material properties. A range of values of the unknown properties can be used to study the relative importance of each parameter. This allows to important to variables to be identified and site investigations can be targeted on them.

Arch assessment is not only concerned with the assessment arch barrels condition but also the assessment of spandrel and parapet wall condition. At present this assessment is entirely subjective, but research is underway into assessment programs able to deal with spandrels.

Bridges are long life structures, which must be expected to be subjected at some time during their service lives to load other demands, which were not foreseen at the time of their design. Some times there are sufficient reserves of strength and durability built into the structure to be able to cope with these unexpected demands but some times there are not. Technical standards may change following new research or public expectations regarding safety may alter, all of which may meat that some existing structure don’t fully comply with the current requirements. Whatever the cause of the change it may not be possible to deal with them in the regular maintenance program and it will be necessary to amount a special exercise to deal with the shortfalls and bring the stock of bridges up to current standards within a reasonable time.

Despite all the guidance and technical advice that can be issued it must be recognized that the management of an inhomogeneous stock of highway structures can be more of and art than an exact science and does require to the engineer to exercise a great deal of judgment. This is particularly true for older structures which use unfamiliar forms of constructions and where detailed aspects of the design are not covered by the current codes. Thus there is often greater uncertainly about dealing with existing structures than there is with design of new structures not always appropriate. There is an example of this in the assessment version of the concrete code where the assessing engineer is allowed to estimate a worst credible strength for materials for use in the calculations, rather than accept a value given in the code.

Chapter 4

WIDENING OF BRIDGE

4.1 GENERAL

With ever improving standards of design and safety, increasing accountability of both bodies, corporate and the individual and the instable demands of road traffic, there is a continuous need to improve the existing highway network. In doing this it is desirable to make as much use as possible of the existing infrastructures.

4.2 INTRODUCTION

Bridge widening, although not new, is probably more a feature of the 20^{th} century than of any other because of the extraordinary growth in the volume and size of vehicular traffic over decades. In considering the need for bridge widening it is probably better to broaden the scope to cover highway widening as this allows the full spectrum of situations which are likely to necessitate the extension of bridges both longitudinally and transversely to be identified and addressed.

This chapter will address these issues by giving a general resume of the possibilities for and limitations of bridge widening or lengthening and by illustrating some of them through specific examples.

4.3 THE REASON FOR WIDENING

The reason for bridge widening may be as a consequence of an overall highway scheme or of a particular problem at the bridge itself. The specific circumstances can be summarized as in the following paragraphs.

1. A highway improvement scheme to,

- Facilitate an upgrade in strategic importance of the route.
- Accommodate a local development.
- Resolve a shortfall in the traffic capacity of the road for its every day (and projected) use.
- Construct a slip road.
- Resolve a local visibility problem by improving sight lines.
- Provide a footway where none exists, due to increasing demand.
- Provide a hard shoulder alongside a carriageway.
- Bring a route up to current standards of design and/or safety.

2. An improvement local to the bridge to,

- Resolve a viability problem arising from intrusive parapets.
- Strengthen sub standard parapets.
- Remove a restriction to the carriageway due to it narrowing over the bridge or due to wall shyness arising from a lack of verge in front of the parapet.
- Remove a discontinuity or narrowing of a footway or hard shoulder at a bridge.

- Accommodate enhanced geometrical standards for improving public safety such as affording necessary clearness and set backs to safety fencing or other obstructions.
- Allow a more economic form of traffic management to be used on a job than the alternative of temporary bridging closure and diversion where that job would not, of itself, require the bridge to be widened.

3. Other examples of highway schemes where bridge widening may be involved are.

- Reconditioning where a structural haunch effectively widening the carriageway, may result in a bridge parapet becoming more exposed to impact or creating a greater intrusion to vision, with the potential for increasing the risk of accidents.
- Widening to enable future maintenance on the route or at a bridge to be carried out without having to close to the road. The restrictions now on minimum lane widths and safety zones for even the most simple of operations are causing an increase in the number of requests for road closures.

4.4 THE WAY OF WIDENING BRIDGES

There are many possible approaches to widen bridges all of which have some merit in particular circumstances. However, widening an existing bridge as opposed to complete reconstructions is not always the wisest move.

Whilst there may be good economical or environmental reasons for doing it, it can equally be a technique where resolution of the problems of achieving a monolithic construction, if required, can be costly, and unsightly. Also structural interaction can cause hidden long-term problems if compatibilities of different forms of design and construction and age of materials are not fully addressed or understood.

The techniques of widening a bridge comprise.

4.4.1 Widening the bridge in like construction to that existing.

4.4.2 Widening the bridge in a different form to that existing.

4.4.3 Over decking or redecking with side cantilevers, on existing substructure.

4.4.4 Redecking on extended capping beam to abutments (and piers).

4.4.5 Construct new parallel road bridge or footbridge.

4.4.6 Revamp lane widths on existing bridge to provide extra lane(s).

The ways of widening are many and varied. A designer needs to be aware of these and the specific problems related to these techniques, at the outset to minimize abortive effort.

4.4.1 Widening in Like Construction

This generally means construction of foundation, substructure and superstructure over the widened section and it can be either symmetrical asymmetrical about the existing bridge.

In the case of arch bridges this technique does afford the possibility of removing existing facing materials for reuse on the new elevation. Whilst this can be a selling point when seeking to do work on a listed building or in a conservation area, such is the subjective nature of these considerations that the possibility of losing the original structure can also kill the proposal. Recent experiences have found conservation planners objecting to modern materials being used in such hidden situations as to saddle an arch because they believe them to be “an intrusion and a fake” and also employing conservation engineers back up their policies by proposing “acceptable” solutions. Conflicts between highway planning, accountability for public safety and conservation are resolvable in an atmosphere of mutual recognition and compromise. In the face of intransigence a stalemate can be reached which does not benefit the highway engineer, the planner, nor ultimately the public they serve.

In the case of beam and slab construction the likely variation characteristics between new and old can be a problem if it is intended to connect the decks structurally. This is especially true the greater the tie between construction dates. Detail considerations must be given to the secondary stresses which are now likely to be experienced by the old deck and what must be done to cope with them. These issues are discussed later, In the event that there is to be no structural connection between the old and the new, and then the butt joint must be under an untrafficked part of the highway to minimize the effects of any differential movement.

4.4.2 Widening in a Different Structural Form

Again this will mean construction of foundation, substructure and superstructure over the widened section and it can be either symmetrical or asymmetrical. However, in this situation the different characteristics of the sections of deck will be of greater significance making structural interaction between the two more difficult to accommodate. In this event the reference to the location of butt joints in paragraph above is especially applicable.

It is unlikely that this approach would be acceptable in an area of environment importance other than with facades incorporating original materials.

Some general points which can be made and which apply to widening in both similar and different forms of construction are.

1. Generally it will be more economical to widen on one side only. Also this makes for a larger, more stable construction especially if there is to be no structural interaction between the substructures.
2. Differential settlement between new and old construction is a real possibility and consequently most widened sections tend to be supported on piles.

3. A decision needs to be made as to the design standards to be employed for the widened section of the bridge. It must be an integral part of scheme such as this that the structural capacity of the original bridge is assessed. The decision will depend on the outcome of this assessment.

4.4.3 Over decking redecking with Side Cantilevers

The essential ingredient of this solution is that the existing substructure is not extended. The original foundations must be capable of accommodating the extra load without detriment to the integrity of the abutments, intermediate piers and original deck if left in place. Obviously, in this respect, an assessment of the substructure is required and, if there are any doubts, remedial action (underpinning and strengthening) must be taken to avoid the likelihood of abortive costs being incurred on the superstructure.

Often, with the benefit of modern materials, it can be arranged that the improved deck weighs not more than the original, This will help the engineer greatly in exercising his judgment as to whether or not to improve or replace the existing substructure.

One common situation where this type of work is undertaken is in the older arch bridges where the mass of the structure itself generally means that any change in overall weight is not significant thus permitting minimum change to the substructure. This approach also enables the original structure and most of its elevation to remain. This is an attraction to conservationists provided that the overhang of the cantilever is not too dominant.

Generally such widening would be symmetrical but care must be exercised in assessing the effects of asymmetric live loads when only one lane is trafficked.

4.4.4 Re-decking on an Extended Capping Beam

Where it is necessary to remove the existing deck then this offers the opportunity to redeck in conventional form spanning between abutments and/or piers across the full width of the deck with all or some of the widening being accommodated on an extended capping beam cantilevering beyond the edge of the abutment or pier. Further widening can be achieved if required by cantilevering the deck. Otherwise the comments in above paragraphs apply equally here.

4.4.5 Construct a New Parallel Road Bridge or Footbridge

In both cases current design standards must be used. It must be borne in mind that along with this it may be necessary to do the strengthening work to the existing bridge, if it has failed its assessment. One crucial decision will be whether to butt the bridges or leave a gap between them. If the later then the gap must be wide enough to access for inspection and maintenance (minimum 1.0m, preferably 1.5m). If butting then the joint must be sealed. This is difficult to do with the inevitable differential vertical movement of the decks. Although separation means changes in alignment of the carriageway and additional runs of parapet, it is generally preferred as all faces are then fully accessible for monitoring and repair. Butted decks are more acceptable if the adjacent vertical faces are shallow as on the end of sloping cantilever parapet beams, so that the edge face of the main body of the deck is accessable.

4.4.6 Provide Extra Capacity by Providing More but Narrower lanes on Existing Bridge Decks

This solution achieves an effective widening rather than an actual widening, It encourages more traffic onto the bridge deck at any one time and the consequent increase in live loading is unlikely to be acceptable when the bridge is assessed. Strengthening may well be needed.

4.5 OVER BRIDGE LENGTHENING

In the case of widening a highway under a bridge then this can be achieved by effectively lengthening the existing over bridge by:

4.5.1 Providing an extra span.

4.5.2 Extending deck to a new abutment.

4.5.3 As far but using techniques such as cable staying or through girders to achieve the extension.

4.5.4 Introducing a new lane or hardshoulder under a sidespan.

4.5.5 By increasing the number of lanes within the existing carriageway.

The fall back situation in all cases is of course full reconstruction.

Generally this is a much more complex affair than widening affecting, as it does, major elements of substructure and, in some cases, the overall articulation of the bridge. Depending upon the age of the existing bridge, lengthening may not be the best solution for long term benefit.

4.5.1 Providing an Extra Span

It is probably the complex of the structural options as it need not affect the overall articulation of the original structure. It can also be designed to current standards, as it is effectively an independent structure. Obviously the existing abutment must be adopted to suit. This could be a useful solution for providing a slip road or a continuous hard shoulder but it would involve lane separation if used as part of the main running carriageway and so is generally to be avoided in this last case.

4.5.2 Extending Deck to a New Abutment

Extending the deck by stitching on an extra length to stretch a span has been done. However, it is complex and does not offer long term benefit as the original design standards will govern the capacity of the overall bridge unless it is feasible to strengthen the remainder of the bridge at the same time.

4.5.3 Techniques such as Cable Staying or Through Girders to achieve the Extension

The options involving cable staying or through-girders to support and extend the bridge overcome the design standard problem as they offer enhanced structural capacity. The later also presents the extra benefit of removing intermediate piers if so desired. However, there is a clear penalty to be aid in aesthetics with the new structures being much more visually intrusive. Also the economics of these types of solution should be compared with that of the more satisfactory new bridge. Avoiding disruption to utilities is an obvious advantage although it is argued later that these should not be in the economic equation.

4.5.4 Introducing a New Lane or Hardshoulder under a Sidespan

Providing a new lane under an existing sidespan avoids affecting the bridge in any way but may require a remaining wall to support the ground in front of the abutment. In this event the requisite safety measures for the inspector when looking at bearing shelf and revetment area must be installed. Also the width of carriageway provided must be adequate to allow its maintenance and of the wall and abutment to proceed without closure.

In all these lengthening options achieving headroom can be a serious problem as bridges are often dipping or sloping down to an abutment and extending this alignment will aggravate the situation. Also the options in paragraph above will intrude into the existing clearance beneath the deck. It will therefore be necessary to consider the possible effects of carriageway lowering and/or deck raising in the cost comparisons.

As mentioned in above paragraph this also could be a useful solution for a slip road or for providing a hardshoulder but would again involve a lane separation if used as part of the main running lanes and so is generally to be avoided in this case. This technique has also been used to provide a high road clearance under a side span, the new lane lowered relative to the existing carriageway.

4.5.5 Increasing Capacity of Existing Carriageway Under a Bridge

Increasing the number of lanes within the existing carriageway generally means loss of any existing hard shoulder or strip. This removes a major benefit in respect of the options available for traffic management and creates hazards during such works, as some of the lanes are generally narrower than normal standard. Furthermore, if new discontinuous hard shoulders are provided then the intruding piers or abutment present an additional hazard to motorists.

4.6 CHANGING DESIGN STANDARDS

It is probably inevitable that, in widening existing bridge; the engineers will seek to carry out the design to current standards. The approach to be used in dealing with the existing bridge will depend upon the type and scale of the widening. If the widening is significant and all on one side then it is possible to accept a new and old concept, always presuming that the original structure has passed its assessment and has adequate provision for abnormal loads. In the event that the original deck has failed to meet these requirements then either the deck can be removed and a whole new deck be provided or the existing deck can be strengthened to either the assessment load or current design standard, preferably the later. If the widening is symmetric then the logic for different standards of deck design diminishes and strengthening or redecking becomes much more likely.

One alternative is to design the extension to the same criteria as the original. This would be difficult to sustain in the light of Highway Authorities accountability for public safety. Although there may be a temptation to do this in quiet rural area, such a location seems at the same time incompatible with the need for widening. Nor can there be certainty over what traffic may use a route in the future especially if there is a need at any time to close an important neighboring route.

Changing design standards generally reflect a growing awareness of how structures and materials react to everyday and long-term use, shortcomings in previous standards and a growing vehicle stock in both size and number. They also challenge the philosophies and sensitivities if engineers who are now charged with condemning bridges they have designed and built. In the climate of accountability and risk, which now exists, engineering judgment may be considered a liability, and certainly there is no tolerance allowed today in exercising it. However it still must play an important part in determining these situations.

In such cases the designer must sometimes consider that, whilst the old is to all appearances sound and fit to continue in use, there is not some fundamental illogicality in adding to it bridge width which has been designed with the benefit of all the latest in technology. He will be well aware that the old section was probably designed to no particular standard, or at best to a similarly applied strips analysis. Perhaps such design as was applied was left to the mason who put it up on the basis that if the arch stayed up when the backed was placed then it would be quite satisfactory for the occasional Horse and cart passage. There are even more troublesome choices. These are between the merits of a 1960's Design, when the belief of the engineer in his ability was absolute and relatively few academic authorities were expounding on design matters, and those of the more sophisticated and lengthier design rules, which have followed in the eighties and nineties.

Where the widening is of an arch by a further arch the engineer can perhaps convince himself that, since design rules for masonry arches still do not carry a great deal of academic weight, it remains acceptable for him to use his experience and judgment in detailing his widened bridge. Where, however, his materials are reinforced or prestressed concrete or steel composite, he will feel less confident that what already exists is fit to be allowed to abut the latest in bridge technology. Indeed, it is often the case that an apparently perfectly functional bridge is effectively discounted from continuing in use by the safety first philosophy that requires any bridge deck which fails its assessment to be strengthened or replaced to the most recent standards.

4.7 VARYING CHARACTERISTICS

Varying characteristics, both structural and material, coupled with changing design standards pose particular problems when seeking to secure structural interaction between new and old construction. Careful consideration must be given to the transverse distribution and torsional effects at the joint and over the old deck. Two specific scenarios exist:

4.7.1 Arch Bridges

- Brickwork and masonry arches have been widened using reinforced concrete fill and overslabbing. The mode of action of an arch with blocks and joints is quite different to that of a reinforced concreted deck slab. One can imagine, and one is encouraged to believe, that the multi-element arch is flexible and can adjust itself to accept live loads and temperature movements throughout its whole barrel, spandrel, parapets and foundation, whilst the slab can only deflect due to load in a vertical direction and expand and contract in the plane of the slab. To fill the arch with rigid unyielding reinforced concrete can be visualized as a sure and certain way to produce problems associated with incompatibility of stiffness and indeed such incompatibility is as much a function of shape and mode of action as of difference in materials.
- However, steps can be taken to separate materials with thin membranes so that they can continue to act dependently, and therefore flexibly. Alternatively, the two materials can be linked that they act homogeneously. There are the undoubtedly theoretical problems which

are created by linking one material with another but it has been done quite successfully for many years and saddling of arches in reinforced concrete is a good example of this process. Fortunately coefficients of passion are not widely different and where interfaces of different materials to occur they are unlikely to be subject to vast changes of temperature.

4.7.2 Beam and/or Slab Bridges

- The problem which arises when it is proposed to widen a deck composed of beams and slab, be they steel or concrete beams, is the likely incompatibility of stiffness between the old and the new. It is suggested that for a long span both the old and the new decks can be idealized and subjected to live loading in some form of modern computer analysis. In this way whatever differences of stiffness exist can be simulated and can be accommodated in the design and detail of the new decks, of the connection between the old and the new and, if necessary, in the strengthening of the old.
- For a short span, if the new and the old deck are substantially of the same form of construction, the differences in deflection will be small but there will nevertheless be a tendency for the stiffer side of the connection to be left supporting the less stiff side. If this less stiff side is the old existing side it is likely that it will see less design load than it should and than it is capable of taking. But the new side can be so designed that it can accept the extra transmitted load thus making the maintenance satisfactory. The reverse is, however, not true and the

transfer of extra load from the new into the old would not be acceptable unless it is economical to strengthen the old deck to accommodate the increased stress.

- Alternatively, the joint between the new and the old can be a butt joint without structural connection. As noted previously, this joint would probably be acceptable in a verge or a central reserve if the edge face is shallow and suitably sealed, but it is not acceptable in the trafficked lane of the carriageway.

4.8 SETTLEMENT AND/OR DIFFERENTIAL SETTLEMENT

Sometimes it is possible to widen a deck physically without widening the sub structure. Sometimes the substructure too must be widened. In either case it is desirable to know to what size and shape and of what material the existing foundation is constructed. Cores can be taken through abutments to establish thicknesses and material properties. Trial pits can be excavated in front of, but rarely behind, abutments to establish depths and widths. Almost inevitably one will find, on carrying out an analysis that the abutment will not stand under modern loading conditions yet it has been standing there for scores of years! What cannot often be determined is whether or not there are timber piles, rafts or mattresses under the foundation and even if these can be identified it is difficult to evaluate their condition and effectiveness.

It has been the case, given that an existing foundation is manifestly working well, that the design engineer can convince himself, and indeed his client that as long as this new deck or his widened deck plus what remain of the existing deck does not exceed the weight of the original, the foundation will be satisfactory for the foreseeable future. As mentioned previously significant savings in weight can often be made by for example replacing stone or brick parapets by steel although for environmental and appearance reasons this may be unacceptable.

The soil beneath an existing foundation will have been compacted and consolidated. The likelihood is that any settlement that was due to take place on that soil under that load will already have happened and that there will be little more to come. The soil under the new foundation will not have been subjected to quite the same forces. When it does become part of the new structure and receives its share of the loading, it will probably settle relative to the existing. The effects of this exhibit themselves simply as a difference in the carriageway levels identified by cracking in the surfacing as ultimately a longitudinal step in the carriageway. But, as already discussed for the deck, it could also be characterized by the transfer of the loads from the new to the old part of the structure, when the subsequence neither the new nor the old foundation is acting as idealized.

One way of overcoming this problem is to construct new foundation and thus eliminate, as far as possible, most of the settlement under the way. The drawback with this is that the final combined structure is left for evermore with a consistent style of foundation which might, over the years, react differently to ground movements, changes in ground water level or any other possible cause that would be better selected by foundation of the same design.

In order to obtain consistency, pilling can be carried out through the existing foundations. This is, of course, expensive, and the designer will begin to wonder whether there is any virtue at all in retaining any of the existing structure. Sometimes when great doubt exists as to the adequacy of a abutment it is useful to build a new substructure behind the existing so as to avoid the need for significant temporary works. This saving would need to be compared with the cost of providing the extra length of span required.

4.9 INSPECTION AND MAINTENANCE

In carrying out a widening scheme due thought must be given to access for inspection and maintenance. In this respect the issue raised earlier of seeking to increase the number of lanes within the existing carriageway, generally by losing a continuous hardshoulder, is a disaster. Where access was difficult before this idea simply creates more have when operational. Where previously it might have been necessary to have only one lane out of two can be closed, to work on bridge, in the changes circumstances it is likely that two of four, will be required with the heavier vehicles being pushed into the narrower outer lanes. This will result in worse congestion than ever. Such an approach is very short and ignores all the lessons, which should be learnt from past experiences.

Also in butting two decks together thought must be given to access for maintenance of the adjacent decks and scaling of the joint. In reality it is probably better not to let the decks touch but to have a small clear gap under the seal to avoid entrapment of debris and potential permanent dampness. Keeping the depth of the face as shallow as possible.

4.10 UTILITIES

Public Utilities are always a major problem area in highway works and widening schemes are no exception. It would be wrong to presume that there is no problem if the existing deck is unaffected. If the utilities were in the verge originally and the bridge is widened they will now be in the carriageway with all the attendant problems that presents for maintenance. Similarly, if they are attached to the outside of the existing bridge they will need to be accommodated in the widened structure. There is a benefit here, as the original eyesore will be removed. Perhaps consideration can be given to providing a separate crossing for the utilities and then declaring the main structure to be protected. Either way a diversion is necessary, adding to the duration and cost of the job.

The cost of utilities works can have a huge influence on the cost of a Job and in some cases can materially affect the outcome in a repair or replace assessment. It is possible that such costs should not be allowed to influence a decision to the long-term disbenefit of the traveling public. The nation has accepted that utilities can be accommodated in the highway and it should be prepared to accept the costs of such a policy. Utility costs should not be part of the economic equation although this could only be practically possible if separate funding is available for them. It is interesting to speculate what will be the situation in the case of future privately built and maintained highways.

Lack of details of the existing structure can present all manner of problems. In particular, the less that is known about a bridge the more costly the investigation will be to uncover the facts or the greater the risk will be of finding something wrong with the old bridge during the contract.

Other influences on the choice of solution are.

- Type and duration of traffic management needed to facilitate the work.
- Nature of what is being spanned and related accessibility.
- Condition of existing bridge.

As mentioned above widening works, especially where the existing deck remains untouched, are not necessarily free of Utility costs. Where they were originally in the verge they might now be in the carriageway. It would be a disadvantage to leave them there, as any maintenance of them will affect the carriageway rather than the verge. However the decision to move them will undoubtedly weigh upon their actual location in relation to available working space and safety zones.

4.11 HIDDEN DIFFICULTIES WITH OLD BRIDGES

Old bridges drawings are very often not totally accurate/available. Our predecessors, even quite recently, did not have the benefits of computer aided draughting or the rigorous document control requirements of a Quality Assurance System to assist them in updating the status of the record drawings. This, together perhaps with many years of accident repairs, maintenance revisions, strengthenings and the attentions of the utility companies may mean that the designer of a widening scheme is not fully aware of what is the precise situation with the part of the structure that is to be retained.

Experience has shown that it is often very difficult to retain parts of an old structure once construction works are commenced. Narrow arch bridges which are due to be widening on one or both sides have been known to give up the struggle when their spandrels are removed to facilitate the works. It is not only the facts that the structures were not formally designed and their stability depends on the live prove whole structure but that the use of modern machinery sometimes lacks the necessary touch and delicacy such works requires. Hand work can be carried out but economics may rule against this.

Recent nationwide concern has concentrated on the condition of postensioned tendons and ducts. There are guidelines for the investigations of these elements. But it is clear that this can only give an indication of the condition of the complete structure and it may only be when the greater accessibility offered by the main works is available that the true extent of any problems can be better seen and evaluated. With the contractor anxious to proceed, decisions to retain existing elements may need to be sacrificed to expediency and economics.

This sort of problems can lead on to the question of how much site investigation should be ordered before the works are designed. The view sometimes taken is that it is undesirable to add to the inconvenience of the road user by carrying out site investigation on the existing structure when the road is going to be disrupted later by the widening works. This is because it is during the works themselves that the better opportunity arises to view the existing condition of the bridge and to verify its capacity to continue in service. The problem of disruption to traffic if major problems are encountered. Whilst a preliminary investigation cannot guarantee that problems will be avoided entirely, or balance, and with careful planning by experiences, it must pay dividends in most cases and is therefore to be recommended.

4.12 CONCLUSION

The consideration involved in any scheme which requires to widen a bridge that exists at present are many and varied. It would be unwise to attempt to make any general rules to apply other than the one to judge each site on its merits. In broad terms the issues to be reviewed fall under the headings of condition and strength of the existing structure, the extent and location of widening required and incidentals. Perhaps the most relevant question relates to the residual life of the existing bridge and the difficulties its premature replacement would cause on the widened highway. Retaining an existing structure is not automatically the right thing to do, nor is knocking it down. But it is very necessary to consider the implications for the future of a decision to keep it.

Assuming that the old can safely be retained, it is probably better not to mix materials or span arrangements with the extension. It will be much more likely that compatibility of deflection, settlement and expansion are achieved if, as far as possible, widening is carried out like for like. It is not usual however, to see arches that have been widened using cast iron or steel beams in some sort of composite action with ordinary fill, or using reinforced concrete are working quite happily. It does seem much less likely however that such solution would be acceptable today not least because of the strong planning and conservation lobby which exists to keep the more extravagant widening schemes firmly in check. Good bridge Engineers will naturally be conservationists at heart but not to the exclusion of all other issues

Chapter 5

CASE STUDIES

WIDENING OF NULLAH LEI BRIDGE NEAR CHAKLALA

5.1 BACKGROUND

Traffic congestion increases in parallel with traffic growth unless a sustained and long term effort is made to overcome the problem. Congestion is grossly inconvenient for all road users, encouraging traffic to divert to less suitable roads, increasing the risk of accidents and reducing safety for the public at large. It also carries an environmental penalty by causing inefficient burning of fuel and increasing the level of exhaust emissions.

The Airport road is a strategic route, linking the main city with the Islamabad International Airport, Chaklala Housing Scheme and 10 Corps Headquarters. The Section between JHANDA and Airport/Chaklala junction of the Airport road carries the highest volume of traffic on both approaches of the bridges over Nullah Lei being one of the factors giving rise to congestion during peak hours in morning (0630 am to 0830 am) and afternoon (1230 pm to 1500 pm). The situation becomes embarrassing when VVIPs are caught in such hold ups. The main users of the route comprises residents of the Chaklala Housing Scheme, persons associated with 10 Corps Headquarters, passengers going to and coming from Islamabad International Airline and the Housing Societies near the Airport vicinity. The large proportion of traffic using the road across Lei Bridge is another factor. During the peak traffic hours because of high traffic density the road used to be blocked for extended duration and the main bottleneck was the Lei Bridge, as all the traffic has to pass over it. Traffic density can be defined as volume of vehicles passing a particular point per unit time.

The bridge falls under the jurisdiction of Rawalpindi Development Authority (RDA). Following steps were undertaken to ease up the traffic congestion:

1. The road geometry (by pass) towards the airport was widened.
2. Traffic signal was installed so as to regulate the traffic flow.
3. Regulatory bodies such as traffic police along with Military police were appointed.

Despite the measures taken as mentioned above, congestion and traffic jams during peak hours continued due to the increase in traffic volume resulting from population growth, thus demanding a solution only in the form of bridge widening.

E-in-C's Branch was directed to carry a feasibility study for widening the existing bridge or construction of a new bridge, to cope with the current and future increase in traffic load. The task was further assigned to DD&C to study the problem and put up a suitable plan together with its financial effect for approval of the COAS.

5.2 DESCRIPTION OF LEI BRIDGE

Lie Bridge was constructed in 1905 for proposed life of 75 years or so. It was designed to British Standard BS153. Along with loading intensities, traffic density had increased considerably since that time. The bridge structure was then required to carry the heavier design loading as well as greater traffic than it was originally designed for.

The bridge comprises of five Arches each spanning 54 feet with an overall length of 270 feet. On top of arches a road is constructed as compacted fill with asphaltic concrete layer of 3 inches. These arches support a 24 feet wide deck, pier are extruding on the both sides by almost 8.25 feet. A general arrangement of the crossing is given in Figure 1.

The superstructure is supported on arches fixed to piers that are extruding outwards on both ends. The piers are on spread footings and are of stone masonry. The structure is placed on the abutment on either end. The present carriageway configuration (illustrated in Figure-1) comprises two lanes i.e. 24 feet wide.

5.3 OPTIONS

A detailed study was carried out to recommend minimizing the traffic congestion and ensure its smooth flow without any interruption. The task was most demanding and challenging, as it included traffic studies, detailed road survey, extension of road including heavy reinforced retaining walls, survey of the existing bridge with a view to consider its feasibility of widening, additional new bridge planning, management and execution, detailed designing and thorough construction planning.

Being the most important administrative route for the Army, the responsibility was taken from the RDA and assigned to the E-in-C's Branch for thorough and efficient planning and for qualitative and early execution. The problem was studied by the Design Directorate with reference to the two options available i.e. widening of the existing bridge and construction of a new bridge. The study led to the following short and long term solutions.

. a. The existing bridge can be widened by an extra 16.5 feet through extension of masonry piers and provision of RCC deck. This will enable the bridge to accommodate 4 lanes each 9 feet wide, and 2 feet 3 inches walkways on either side. The cost of remodeling work was estimated at Rs. 3.0 million.

b. In long term, a permanent bridge downstream of the existing bridge should be planned. The cost of new RCC Bridge was estimated to be Rs. 28.00 million.

The conferences at different forums were held and as a result, presentation on the widening of Lie Nullah Bridge was given to Additional Secretary (Defence), who gave the following decisions.

2. Widening of existing bridge was approved as a short-term measure. In addition, it was decided that two additional turn loops at the T-Junction towards airport side shall be added to relieve the pressure of traffic on the junction (see attached sketch). Survey of the area was undertaken to evolve the geometric and pavement structure design of these turn loops.

3. To solve the problem permanently, construction of a new bridge was approved. However it was decided that the new bridge should form part of an overall traffic regulation plan from the airport to the Mall near District Courts.

Soil exploration in the vicinity of piers/abutments was not permitted, as it was feared that it would cause adverse effects on the strength. However visual inspection was carried out, which revealed no evidence of settlement.

5.4 OPTIONS ADOPTED AS A SHORT TERM MEASURE

Considering the immediate requirement, resources, funds and time, above was accepted and the work was undertaken on emergent basis as a short term measure with the instruction to process construction of new bridge also. The work on both the options started in May 1994 and accordingly widening was completed in December 1994 whereas the new bridge was opened to traffic in year 2001. The designing responsibility was given to the DD&C (E-in-C's Branch) for not only extension of bridge but also widening of road astride the bridge including filling and heavy retaining wall reinforced structure. DD&C experts took keen interest, and due to their dedication, devotion and experience were able to complete the designing with in three weeks. Execution phase was also monitored by the experts.

The decision was taken to widen the bridge on both sides by 8.25 feet each giving a total extension of 16.5 feet. The 12 feet to be utilized for the roadway and 4.5 feet for the walkways on the both sides. The extruded portions of the piers were elevated by 11.5 feet and precast prestressed girders were placed on them. Reinforced concrete slab only over the widened portion was placed and was joined with the existing deck. Finally, resurfacing with asphalt mix was done over the complete roadway (i.e. new and existing deck) and bridge was successfully opened to the traffic.

The new carriageway configuration comprises of four lanes i.e. 36 feet wide. Four lanes for vehicles (9 feet each) and walkway (2 feet 3 inches) on each side.

5.5 WIDENING CONSIDERATIONS

Before the widening of bridge, design and construction engineers considered several potential problems. The main considerations were retention of existing bridge elements, traffic control, structural constraints, and construction limitations.

There were also certain elementary procedures that were addressed for the bridge.

a. Record/drawings were not available and specifications of the original structure were reviewed thoroughly.
b. A thorough inspection of the existing structure was performed and no unusual/alarming symptoms were observed in site conditions.
c. No additional subsurface information, including borings could be obtained due to the fear that it might not damage the pier foundation.

The first consideration for bridge widening was whether to retain those parts of the existing deck those are structurally sound. Considerations were given to replacing the entire existing deck but as remaining old deck was more than half of the new deck i.e. two-third width and neither the existing deck was severely deteriorated, so ultimately it was decided to retain the existing deck.

As the existing deck was to be retained, the design for moment and shear transfer through the longitudinal joint between the new and existing portions of the deck was provided. The arrangement is shown in the Figure 4, whose details are as under.

a. Half inch diameter bars provided in existing edge beam at 18” c/c as stirrups were welded to the bottom reinforcement of the deck slab.

b. While modeling the slab girder bridges, minimum slab thickness as per AASHTO is 7.5 inches so as to ensure that deck slab is stiff enough to transfer the external loads on the girders, which in this case was kept 8 inches.

5.5.1 Maintenance of Traffic

A prime concern was the safety and convenience of the traveling public, the safety of construction personnel, and potential damage to the work. Another consideration was the effect of the widening upon the safety of the public using the space beneath the bridge and any traffic related impact that the widening may have on that roadway

As there was no convenient alternate route that may be used as a detour during bridge widening, therefore traffic was allowed to use the existing bridge during widening. This created congestion at the work site and resulting vibrations and deflections from live loads were to be catered in design and execution of the widening work.

5.5.2 Differential Superstructure Deflections

Differential deflection between new and existing superstructures is not a problem if the joint between the two occurs in a median or untravelled area. Generally, in such cases the superstructures are not connected. Joint should be located out of the travelled lanes whenever possible, but most frequently and also as in this case the joint between new and existing decks occurred within the travelled way and the two were structurally connected.

5.5.3 Vibrations from Traffic

Traffic induced vibration has been blamed for distress occasionally observed in new construction that connects to structures carrying live loads. Research indicates that such damage is relatively rare and can be eliminated by the use of proper construction sequence and correct design details, which was ensured.

5.6 GENERAL DESIGN CONSIDERATIONS

There were certain aspects of structure type selection, framing considerations and design details that are unique to bridge widening. However, the widening accomplished solely on the superstructure, and balanced i.e. symmetrical on both sides was adopted.

5.7 SELECTION OF STRUCTURE TYPE

Many factors influence the designer's choice of type of structure for widening. The natural choice is to use the same type as the original, but this is not mandatory. Some of the factors that influence the choice of structure type for a widening are:

5.7.1 Aesthetic and Historic Considerations

Aesthetic and historic factors favoured the maintaining of the original appearance of a bridge. This does not necessarily meant that the same structure type should be employed. Multani tiles were fixed on the railing side towards the deck. To all but the very astute viewer, the architectural integrity of the original design was not altered. The widening was accomplished in a manner such that the existing structure does not look, add on to.

5.7.2 Roadway Geometry

The widening consisted of almost doubling the bridge width (i.e. two lanes to four lanes divided). The by pass towards the airport was widened by providing a new road towards left before the traffic signal. A huge earth filling and heavy retaining walls were constructed for the purpose. Also road astride the bridge was widened. As vertical clearance beneath separation structures was insufficient to allow for false work during construction of a widening the use of precast prestressed concrete girders was decided.

5.7.3 Maintenance of Traffic

The problems associated with maintaining traffic includes the safety of the public and the workmen. As the detour was not feasible and traffic was to be carried through the work area, proper sequencing of construction operations was essential. Much of the work as possible prior to the removal of the existing curb and railing was done rather entire widening was completed, including making connection between old and new decks before removing the existing rails.

As the heavy volume of commuter traffic prevented closing the existing bridge lanes except during brief off peak periods each day, special measures were needed. Most of the work on the bridge was carried on during night. For the placement of the girders by crane and for laying the asphalt the bridge was closed at night with the help of district management authorities.

5.7.4 Deflection Characteristics

Deflection characteristics were taken into account when the new deck was to be rigidly connected to the existing deck. The designer considered the relative deflection characteristics of the existing and new portions of the bridge and then selected the type of structure for widening. As there was no deflection in the existing deck, the deflection of the new deck was minimized by taking flowing steps:-

a. Thickness of the new deck was kept 8 inches.
b. Rigid joint between the new and existing deck was provided.
c. In total, the new deck slab was supported at three places i.e. by existing beam and two girders (details as per Figure 4)

5.7.5 Differential Expansion Characteristics

Expansion joints are provided to control the cracks caused by contraction and expansion of RCC due to thermal changes. The transverse deck joints were located in the superstructure of the widening at the top of the piers.

5.8 DESIGN AND CONSTRUCTION DETAILS

Standards used for new bridge construction were used for widening. These include the AASHTO Standard Specification for Highway Bridge and the various ACI documents listed in the Recommended References. Some construction operations that are unique for widening are discussed hereunder.

5.8.1 Concrete Removal

As portion of the bridge was to be removed, during the removal operations it was ensured that no damage is caused to any portion of the structure that is to remain in place. Existing reinforcement that was to be incorporated in the new work was protected from damage and was thoroughly cleaned of all adhering material before embedding in new concrete. Before beginning concrete removal operations involving the removal of a portion of a monolithic concrete element a saw cut was made taking care to avoid the reinforcing steel, to a true line along the limits of removal on all faces of the element which will be visible in the completed work.

5.8.2 Refinishing Exposed Areas of The Deck

Concrete exposed by rail, curb or sidewalk removal was too rough to serve as a riding surface. A bituminous overlay was placed for refinishing.

5.8.3 Traffic Induced Vibrations

As the vibrations are carried into freshly placed concrete through reinforcing steel extending from the existing bridge, damage to new concrete could occur. Such damage was avoided by attaching the forms to the existing bridge and traffic control.

When a reinforcing bar moves relative to the concrete, the displaced concrete will readily flow back and forth with the bar, as long as it is still in the original state. But as initial set begins only weak water diluted grout flows back to surround the bar. Also cracks may develop in the plastic concrete and fill with weak material along a horizontal plane with adjacent bars along sloping planes running from the bar to the surface of the deck. This condition may result in a severe reduction in bond to reinforcement and premature deck spalling. Similar damage may occur in new bridge decks if live loads from workmen or equipment are allowed directly on poorly blocked up reinforcing steel on the outside of the construction joint bulkhead. For this reason, workers and equipment near the perimeter of a reinforced concrete deck during placing and finishing operations were restricted to planks blocked up from the forms, rather than bearing directly on any reinforcing steel that extends through bulkheads and into the concrete being placed. Although it would seem that any movement of reinforcing steel extending from a structure carrying traffic into freshly placed concrete on a widening would result in the defects described, certain practices will generally eliminate such damage. The practices which were employed when concrete was placed against an existing structural element carrying traffic include the following.

5.8.4 Reinforcing Details

The reinforced concrete slab, eight inch thick was placed over the girders.

- Half inch diameter seven wire strand prestressing cables were provided laterally through the deck at middle third of each span (at the position of diaphragm in longitudinal plane). Centroid of cable is half inch below the centroid of deck.Five kips of prestressing force was applied to each cable.

- Half inch diameter bars provided in existing edge beam at

  18” c/c as stirrups were welded to the bottom reinforcement of the deck slab.

5.8.5 Forming Details

The forms for the widening were connected to the existing bridge during placement of concrete for widening. To control differential deflection caused by traffic following measures were taken:-

1. Diaphragms between adjacent girders, a rigid temporary blocking system was used to equalize girder deflection until the deck slab gained sufficient strength. Sometimes the forming system itself offers sufficient rigidity.

2. A smooth-riding surface was maintained on the deck and the approach roadway and a good grade match was obtained where they join.

3. Traffic speed and/or allowable loads were existing bridge during and immediately after placing new deck concrete.

4. The traffic lane adjacent to the connecting joint was closed for a few days after placing new deck concrete.

5.9 EXECUTION SEQUENCE

After removal of the railing and walkway the different steps of execution are as under:-

5.9.1 Elevation of the extruded piers.

5.9.2 Placement of the girders.

5.9.3 Construction/Joint of the decks.

5.9.4 Resurfacing with asphalt mix.

5.9.1 Elevation of the Extruded Piers

Concrete of 3000 psi strength at 28 days on 6 inch cubes and grade 40 deformed bars was used. The overall width and breadth of the extended pier is 5 feet and the total elevation given is 11.5 feet. Sloping PCC over the type of power extending beyond existing deck width was removed and the surfaced leveled before the installation of dowel bars and RCC pier. The reinforced concrete structure was anchored at the bottom as well as the sides by the anchoring bolts as shown in the figure. Details are given as per Figure 3.

5.9.2 Details/Placement of Precast/Prestressed Girders

- Design is based on requirements of MLC 70 loading.
- <u>CONCRETE STRENGRTH</u>

  a. Material Strength i.e. f'c =5 Ksi.

  b.Concrete Strength at the time of post tensioning was 4 Ksi.

- <u>STEEL</u>

  a. Reinforcing Rebars, stirrups etc, in girder were grade 60 deformed bars.

  b.Pestressing steel conformed to ASTM 416 with minimum ultimate strength of 270,000 psi and consisted seven wire strands half inch in diameter, each strand having a minimum area of 0.153 inch.

- Design is based on multi strand system, each multi strand comprising of seven monostrands of half inch diameter, each mono strand of seven wires, all laid in a conduit of minimum internal diameter of 2.1 inch.

- Steel duct (3.25 inch diameter) used was corrugated galvanized semi-rigid conduit of 24 gauge thickness and was rigidly supported by ties to reinforcing steel at 4 feet distances.

- Cables were stressed after concrete had attained a strength of 4,000 psi.

- Order of prestressing from one end only; no.2 cable was jacked from end opposite to that from which no. 1 is jacked.

- Maximum jacking force per tendon was 202 kips.

- Cables were grouted with neat cement grout of 0.55 water cement ratio. Anchorage recess was filled with 3 ksi concrete after grouting and trimming cables. 5 grams of Al powder per kg of cement was used as expansive agent for grouting.

- To ensure placement of girders without damage and accident girders were lifted by crane through slings tied at end blocks of the girder.

- After placing the girders on bearing pads they were temporarily braced laterally until diaphragms and deck slab were casted.

- The surface of prestress girder at top was roughened for proper shear transfer.

- Diaphragm was provided at end and third point of the girder,one inch dia conduit was provided in the girder and diaphragm for passing lateral tie.
- The girders and diaphragm were tied together after launching through lateral tie.
- One inch dia conduit was left at the end of each girder for the horizontal prestress of all four girders on a span for lateral stability.
- Half inch dia seven wire strand cable was passed through the conduits and stressed to twenty kips.
- Prestressing arrangements.

    a. The arrangements provided at the end of each girder for prestressing was utilized.

    b. Two half inch dia seven wire strand cables passing through top hole at the end of girders and each was prestressed to nominal prestressing forces of four kips.

    c. At the end of girders central and bottom conduit were used for tie through the nut bolt arrangement.

The girders were lifted and placed over the pier with the help of crane. No 6 grade 40 bars, zinc coated initially tied the girders with one another till construction of diaphragm.

5.9.3 Construction/Joining of Deck

The reinforced concrete slab 8 inch thick was placed over the girders.

a. Half inch dia seven wire strand prestressing cable were provided laterally through the deck. At middle third of each span (at the position of diaphragm in longitudinal plane). Centroid of cable is half inch below the centroid of deck 5 kips of pre-stressing force was applied to each cable.

b. Half inch dia bars provided in existing edge beam at 18' c/c stirrups were welded to the bottom reinforcement of the deck slab.

Communication trench dug for passage of signal cables was back filled with coarse material. Out let for accidentally trapped water/moisture in the trench was provided at every 10 ft interval from the abutment.

5.9.4 The resurfacing was done by the NLC Engineers

5.10 STRUCTURAL ANALYSIS

1. DATA & SPECIFICATIONS

Clear Span	=	54 ft
Clear width	=	36 ft

System of Loading = HS – 20

Area of Prestressed

Steel, Aps = 2.142 sqin(2xTendons of Seven ½” dia Strands each)

Area of Non Prestressed

Steel, As = 2.48 sqin

(8 # 5 bars)

Girder, fy = 60,000 psi

Concrete Str of Girder,fc = 5000 psi

Concrete Str of Slab,fc = 3000 psi

Deck, f y = 40,000 psi

(Grade 40 reinforcement)

Ultimate strength of Tendons,

fpu = 270’000 psi

GIRDER PROPERTIES

a. Area of concrete, Ac = 408 sqin

b. Moment of Inertia, Ic = 70516 in 4

c. Effective depth of prestress tendons,

dp = 35.5 in

d. Effective depth of nonprestress steel,

	d	=	35 in
e. Effective width,	b	=	15 in
f. Average width of flange,	hf	=	8 in

2. SLAB ANALYSIS

Clear Span = 12 in

Thickness of Slab = 8 in

Thickness of wearing coarse = 3 in

Effective depth,d = 6.5 in

Area of Steel, As = 0.59 sq in

Live Loading = HS 20 (system of loading)

Dead Ld BM

Wt of slab		=	8/12x150=100 psf
Wt of wearing coarse		=	3/12x138=34.30 psf
Possible future covering		=	15 psf
Total D.L	wo	=	150 Psf

$$BM\ (D.L) = \frac{woL2}{10} = \frac{150(1)\ 2}{10} = 15 \text{ lb-ft}$$

Live LD BM

When Main reinforcement is perpendicular to traffic the live load moment for HS 20 Loading.

$$= 0.8 \times \frac{S+2}{32} p20 = \quad \text{moment, lb-ft per foot width of slab}$$

$$BM\ (LL) = 0.8 \times \frac{3}{32} \times 16000 = 1200 \text{ lb-ft}$$

As the loaded length is small, the impact coefficient is 0.30.

=> BM (L.L) = 1200x1.3 = 1560 lb-ft

BM Total = BM (D.L)+BM (L.L)

= 1575 ft-lb

MOMENT OF RESISTANCE BY THE SLAB

Moment capacity of Slab is given by

$$M = As\ fy\ (d - a/2)$$

Where, $a = As\ fy/0.85\ fc'b$

$a = 0.59 \times 40/0.85 \times 3 \times 12$

$= 0.77$ inch

Now $M = 0.59 \times 40000 \times (6.5/12 - 0.77/2)$

= 12025 **ft-lb**

Design Moment = 0.9 x 12025

10822 > BM req (1968.75 ft-lb)

Hence OK

ANALYSIS OF PRESTRESSED GIRDER

Note: - When system of wheel loads move as a unit across a beam , the bending moment is max under one of the loads. To determine the position of loads when the moment is max under a particular load the system of loads must be in such a position that the central line of the span is midway between that load and the resultant of all loads on the span. With the loads in this position, the reactions are computed and equilibrium equations are applied to compute the Bending moment in the beam under the particular load. (Strength of Material by F.L.SLunger)

Each Girder must support **One** wheel load per wheel.

a. <u>CASE 1</u>

When Pt A and Resultant R are at equidistant from center line

Taking Moment at R2

R1 x 54 =36000 x 17.66

=> R1 = 117755 lbs

Moment at 'A'

B.M at A = R1 x 17.67

= 208073.85 **lb-ft**

b.<u>CASE II</u>

When Pt B & Resultant 'R' are at equidistant from center line

Taking moment at R2

R1 x 54 = 36000 x 24.67

R1 = 16447 lb

Moment at B

= R1 x 24.67 - 4000 x 14

BM at B = 349739 lb-ft

b. <u>CASE III</u>

Pt 'C' and Resultant 'R' equidistant from center line

Taking Moment at R2

R1 x 54 = R x 31.665

R1 =21110 lb

R2 =14890 lb

Moment At 'C'

= R2 x 22.33

BM at C = 332643 lb-ft

A comparison of the proceeding results shows that the most dangerous bending moment is in case II.

I.e. BM max = 349739 lb-ft

<u>IMPACT COEFFICIENT IS</u>

$$\frac{54}{54 + 125} = 0.301$$

Impact moment = 105508 Ft-Lb

BM max = 455.25 K Ft

MOMENT AT 10 feet from the SP

M10 = R1 x 10

= 16447 x 10

= 164470 lb-ft

Adding Impact Coefficient

M10 = 213976 lb-ft

MOMENT AT 20 feet FROM THE SP

M20 = R1 x 20-4000 x 9.33

M20 = 291620 lb-ft

Adding Impact Coefficient

M20 = 379398 lb-ft

BENDING MOMENT DUE TO DEAD LOAD

The wt of the slab per foot of beam

= 150 psf

The wt of girder = 408/144 x 150

= 425 psf

The dead load moment at center of span

Md = 1/8 x575x(54)^2

= 209587 lb-ft

= 209.5 K-ft

Dead load moment at 10 ft from SP

M10 = R1 x 10 - (575 x 100/2)

= 126500 lb-ft

= 126.50 K-ft

at 20’ from Sp

M20 = R1 x 20 - (575 x (20)2/2)

= 195500 lb-ft

= 195.5 K-ft

<u>TOTAL BM i.e. Live + Dead</u>

BM max = 665 K-Ft

M10 = 340.5 K-Ft

M20 = 575 K-Ft

MOMENT OF RESISTANCE BY THE GIRDER

Moment of Resistance by the section with fpu = 270,000 psi the stresses in steel at design load in approximately

$$fps = fpu\left(1 - \frac{0.5\ e\ fpu}{fc'}\right)$$

Steel ratio, e = Aps/ bd

$$= \frac{2.142}{15 \times 35.5} = 0.00402$$

$$fps = 270\left(1 - \frac{0.5 \times 0.004 \times 270}{5}\right)$$

$$fps = 240.6757 \text{ Ksi}$$

To check whether stress block depth ‘a is less on greater than the average flange thickness.

$$a = \frac{Aps\ fps}{0.85\ f'cb}$$

$$= \frac{2.142 \times 240.67}{0.85 \times 15} = 8.1 \text{ in} > \text{avge flange thickness}$$

Therefore sec acts as a flanged sec

FOR FLANGED SEC WHEN NON PRESTRESSED STEEL IS PRESENT

$$Mn = Apw\ fps(dp-a/2) + Apf\ fps\ (d - hf/2) + Asfy\ (d - a/2)$$

Where

$$Apw = Aps - Apf$$

$$A_{pf} = \frac{1}{f_{ps}} (0.85\, f_c'(b - b_w)\, h_f)$$

$$= \frac{1}{240.67} (0.85 \times 5\, (15-7)\, 8)$$

$$= 1.13 \text{ in } 2$$

$$A_{pw} = 2.142 - 1.13$$

$$= 1.012 \text{ in } 2$$

Now

$$a. = \frac{A_{pw} f_{ps} + A_s f_y}{0.85\, f_c'\, b_w}$$

$$= \frac{1.012 \times 240.67 + 2.48 \times 60}{0.85 \times 5 \times 7}$$

$$= 13.188 \text{ Say } 13.2 \text{ in}$$

Mn = Apw fps (dp – a/2) + As fy (d – a/2) + Apf fps (d – hf/2)

= 1.012 x 240.67 (35.5 – 13.2/2 + 2.48 x 60 (35 – 13.2/2) + 1.13 x 240.67 (35 – 8/2)

= 19695.5 K-in

= 1641 K-ft

Design Moment = **1477 > 665 K-Ft**

Hence Ok

FOR SHEAR

a **Dead Ld Shear**

The max dead ld shear at the end of the beam

S max = 27 x 575 x 1.4 = 21.735 K

S10 = 13.685 K

S20 = 5.635 K

b. **Live Load Shear**

The absolute max shear occurs with the truck on the span in the position shown in fig.

Max live ld shear = 29.8 K

Max live load shear at 10,20 and 27 are

S10 = 23.1 K

S20 = 16.45 K

S27	=	11.85 K		

Impact shear at

S max	=	8.32 k		
S10	=	6.45 K		
S20	=	4.59 K		
S22	=	3.31 K		

Total live load shear

S max	=	38.12 K		
S10	=	29.55 K		
S20	=	21 K		
S27	=	15.16 K		

Total shear is here as under:-

S max	=	60 K	=	147 psi
S10	=	43.25 K	=	106 psi
S20	=	26.635	=	65 psi
S27	=	15.16 K	=	37 psi

Max design shear Vu, at distance “d” from Sp

Vud	=	49.6 K	=	122 psi

<u>SHEAR RESISTANCE</u>

Shear Resisted by concrete.

Vc	=	o.95x(f’c)^1/2	=	0.95x(5000)^1/2

Vc = 67 psi

Vc = 0.85 x 67 = 57 psi

As,Vu= (Vc + Vs)

Vs = Vu - Vc = 122 - 57 = 65 psi

Vs = 77 psi

Shear Resistance by Stirrups

a. From Sp to 5 ft # 4 bars @ 6” C/C)

$$Vs = \frac{Av\ fyd}{S}$$

$$= \frac{0.78 \times 40 \times 35 \times 1000}{6 \times 408} = 379 \text{ psi} > 77 \text{ psi}$$

Ok

b. From 5’ to 15’

4 bars @ 8.5 “ C/C

$$Vs = \frac{0.56 \times 40 \times 35 \times 1000}{8.5 \times 408}$$

= 226 psi > 77psi **Ok**

C. <u>From 15’ to 27’</u>

4 bars @ 14” C/C

$$Vs = \frac{0.34 \times 40 \times 35}{14} = 34\ K$$

$$= 83\ psi > 77psi\ Ok$$

WIDENING OF SOAN BRIDGE

5.11 BACKGROUND

Grand Trunk road is a strategic route linking Lahore city with the Peshawar city via Rawalpindi. The Soan River crosses the GT road just on the out skirts of the Rawalpindi city. Due to the extra ordinary growth in the volume and size of vehicular traffic it was decided to widen the existing GT road dual lane single carriageway way to dual lane double carriageway. The portion between the Rawalpindi and Kharian for widening was given to the NLC Engineers by NHA. This portion is of 162 Km long including numerous bridges. Most of the bridges were used as such but two bridges were widened in this portion as well. One was the bridge over Sohowa Nullah, which was widened by NLC Engineers from 6.7 meters to 9.5 including walkway and railing on both sides in 1997. Second was the bridge over Soan River, which was widened by the NLC Engineers. The work on the site commenced in August 2000 and was completed by July 2001.

5.12 DESCRIPTION OF SOAN BRIDGE

Soan Bridge was constructed in 1905 for proposed life of 75 years or so. The design was based on British Standard BS153. Along with loading intensities, traffic density had increased considerably since that time. The bridge structure was then required to carry the heavier design loading as well as greater traffic than it was originally designed for.

The bridge comprises of thirteen Arches each spanning 50 feet with an overall length of 650 feet. On top of arches a road is constructed as compacted fill with asphaltic concrete layer of 3 inches. These arches support a 22 feet wide deck. A general arrangement of the crossing is given in Figure- 5.

The superstructure is supported on arches fixed to piers. The piers are on spread footings and are of stone masonry. The structure is placed on the abutment on either end. The present carriageway configuration (illustrated in Figure-6) comprises two lanes i.e. 22 feet wide.

5.13 OPTIONS

A detailed study was carried out to analyse the options. The task was the most demanding and challenging as it included, survey of the existing bridge with a view to consider its feasibility of widening, detailed designing and thorough construction planning. As a result it led to following options.

c. The existing bridge can be widened by an extra ten feet through redecking with side cantilevers. This enabled the bridge to accommodate two lanes each 13.5 feet wide, and 2 feet walkways on either side.
d. Construction of new bridge should be planned.

Option ‘a’ was adopted being less cost effective.

5.14 WIDENING CONSIDERATIONS

For Rehabilitation/widening a significant amount of time was spent in the field to properly evaluate the present condition of all structural elements by the Engineers.

Following conclusions were drawn from the detailed investigations:

- Rehabilitation of the Bridge is feasible and practical.
- All bridge elements can readily be upgraded to support all of the AASHTO live loadings.
- The basic concrete arches are reasonably sound and after removal of roadway pavement, fill, and spandrel walls, can be rehabilitated and capped with a continuous strip of reinforced concrete. Other rehabilitation measured would include introduction waterproofing membrane in conjunction with new spandrel wall construction; installation of replacement fills, road, and sidewalk pavement; and total replacement of all drainage systems.
- Pier and abutment structures are adequate and can simply be rehabilitated.
- Introduction or upgrading of a number of features relating to pedestrian, vehicular, operational, and life safety to ensure compliance with modern standards is required.

Before the widening of bridge, design and construction engineers considered several potential problems. The main considerations were retention of existing bridge elements, structural constraints, and construction limitations.

There were also certain elementary procedures that were addressed for the bridge.

d. Record/drawings were not available and specifications of the original structure were reviewed thoroughly.
e. A thorough inspection of the existing structure was performed and no unusual/alarming symptoms were observed in site conditions.
f. No additional subsurface information, including borings could be obtained due to the fear that it might not damage the pier foundation.

The first consideration for bridge widening was whether to retain those parts of the existing deck those are structurally sound. Considerations were given to replacing the entire existing deck and finally it was decided to replace the existing deck.

As the existing deck was to be replaced, the designing of the deck was required. The arrangement is shown in the Figure 4, whose details are as under.

5.14.1 Maintenance of Traffic

As, there was convenient alternate route in the form of a newly constructed bridge, which was used as a detour during bridge widening.

5.15 GENERAL DESIGN CONSIDERATIONS

No site investigation of the substructure was undertaken except for a visual inspection, which revealed no evidence of settlement. The basis of decision in favour of retaining the existing substructure was that the weight of the new deck was to be less than the old. In simple terms this consideration was satisfactory.

The problem, which was introduced by the decision, was that the articulation of the new deck applied a horizontal force due to braking which, in theory, the existing brick abutments would not be able to accept. The solution to this problem was to take this load via a approach slab into the fill and road construction behind the abutments. This extension slab also overcomes the problems of achieving the widened bridge over the wing wall area and a cantilever on both ends for walkway and railing.

The essential ingredient of this solution is that the existing substructure is not extended. The original foundation was capable of accommodating the extra load without detriment to the integrity of the abutments and intermediate piers.

Often, with the benefit of modern materials, it can be arranged that the improved deck weighs not more than the original. This helped the engineer greatly in exercising his judgment to replace the existing substructure.

Hence extension of the existing bridge by providing one additional lane was carried out which included an overall widening of 1.75 meters in roadway and 1.3 meters for the walkway and railing.

There were certain aspects of structure type selection, framing considerations and design details that are unique to bridge widening. However, the widening accomplished solely on the superstructure, and was balanced i.e. symmetrical on both sides was adopted.

5.16 SELECTION OF STRUCTURE TYPE

Many factors influence the designer's choice of type of structure for a widening. The natural choice is to use the same type as the original, but this is not mandatory. Some of the factors that influence the choice of structure type for a widening are:

5.16.1 Aesthetic and Historic Considerations

Aesthetic and historic factors favored the maintaining of the original appearance of a bridge. Brick tiles were fixed on the outward side of the railing. To all but the very astute viewer, the architectural integrity of the original design was not altered. The widening was accomplished in a manner such that the existing structure does not look, add on to.

5.16.2 Roadway Geometric

The road towards the Rawalpindi was raised by almost four feet . A huge earth filling was carried for the purpose. Also road astride the bridge was widened.

5.17 DESIGN AND CONSTRUCTION DETAILS

Standards used for new bridge construction were used for widening. These include the AASHTO Standard Specification for Highway Bridge and the various ACI documents listed in the Recommended References. Some construction operations that are unique for widening are discussed hereunder.

5.17.1 Reinforcing Details

The reinforced concrete slab with side cantilevers, eight inch thick was placed on top which was supported at the ends by wing walls and in between throughout by sand fill.

5.18 EXECUTION ON SITE

5.18.1 Deck Removal.

5.18.2 Maintenance/strengthening of the bridge.

5.18.3 Redecking with Side Cantilevers.

5.18.4 Resurfacing with Asphalt Mix.

5.18.1 Deck Removal

As complete deck of the bridge was to be removed, during the removal operations it was ensured that no damage is caused to any portion of the structure that is to remain in place.

5.18.2 Maintenance/Strengthening of the Bridge.

Earth filling and masonry walls of the old bridge were removed and the top surface of the arches brushed cleaned and all the cavities were filled with cement grout and than 75 mm RCC BAND on top of arch was applied. The visible cavities in the soffit of arches were repaired with cement and mortar (1:2 mix) and than were plastered with cement mortar (1:3 mix).The bottom portion of the piers was repaired by filling the cavities and then plastering.

5.18.3 Redecking with Side Cantilevers

Sand filling between the wing walls was provided and lean concrete on the top of sand fill was laid. Over it deck slab with side cantilevers was poured. Grade 60 reinforcement was provided in the deck slab, arch bands and walkways.

5.18.4 Resurfacing with Asphalt Mix.

A three inch layer of asphalt was laid on top of deck.

Chapter 6

RECOMMENDATIONS/GUIDELINES FOR WIDENING OF EXISTING HIGHWAYS BRIDGES

6.1 INTRODUCTION

Many highway bridges become functionally obsolete due to inadequate width long before they become structurally deficient. Since widening almost always is more economical than complete replacement, there is a need to make available the results of research and field experience pertaining to the widening of the bridges. This research discussed many problems that are unique to the widening of existing concrete/Arch bridges and bridges with concrete decks. The primary focus of this document is on bridge decks. However, substructure issues are also raised and discussed. The effects of differential movements between the existing and new portions, including movements due o traffic on the existing structure during construction, are discussed.

General recommendations are made pertaining to the choice of structure type, design details, and construction methods and materials.

6.2 COMMON WIDENING CONSIDERATIONS

When a bridge is to be widened, several potential problems should be considered by design and construction engineers. Some of these considerations are retention of existing bridge, elements, traffic control, structural constraints, and construction limitations.

There are also certain elementary procedures that should be addressed for all structures.

1. Review the record drawings and specifications of the original structure.
2. Review any change orders that might have been approved during the original construction.
3. Perform a thorough inspection of the existing structure and note any changes in site conditions, such as bank scour.
4. Obtain additional subsurface information, including borings.

The first consideration for bridge widening is whether to retain those parts of the existing deck those are structurally sound. Consideration should be given to replacing the entire existing deck if the remaining old deck will become less than half of the new deck width and/or the existing deck is severely deteriorated.

If the existing deck is to be retained, the design should provide for moment and shear transfer through the longitudinal joint between the new and existing portions of the deck.

6.2.1 Maintenance of traffic

Prime concerns are the safety and convenience of the traveling public, the safety of construction personnel, and potential damage to the work. Another consideration is the effect of the widening upon the safety of the public using the roadway beneath the bridge and any traffic related impact that the widening may have on that roadway.

Ideally, there should be a convenient alternate route that may be used as a detour during bridge widening operations, so that all traffic may be kept off the bridge however, this is seldom the case. Due to the high cost of temporary detour bridge, economy usually dictates that traffic be carried on the existing bridge during widening. This creates congestion at the work site and resulting vibrations and deflections from live loads on the existing bridge may affect concrete in the new work.

6.2.2 Prevention of Damage to Existing Structure

Bridge widening generally involves shored excavation immediately adjacent to existing bridge and removal of portions of the existing bridge. Shoring of excavations is usually the responsibility of the contractor. Since public safety and safety of the existing bridge or adjacent highway facilities can be jeopardized by failure of shoring, construction engineers should carefully monitor this phase of the work. Specifications should require that shoring be designed and inspected by licensed engineers. Designers should minimize lengths and widths of excavations.

6.2.3 Differential Foundations Settlements

The amount of differential foundations settlement that can be tolerated between old and new construction depends upon the configuration of the widening. If the joint between the existing structure and the new structure is outside of the traveled way (i.e. in the median), or rigid attachment of the widening to the existing structure is to require for overall stability, the existing and new structures may be left unconnected and differential settlements tolerated. However, it is usually necessary for the new foundation to be compatible with the existing. This will mean that the new foundation must be designed to settle very little, dictating piles or drilled shafts, unless rock is present near the surface.

6.2.4 Differential Superstructure Deflections

Differential deflections between new and existing superstructure are not problem if the joint between the two occurs in a median or untraveled area. Generally, in such cases, the superstructures are not connected. Joints should be located out of the traveled lanes whenever possible, but most frequently the joint between new and existing decks does occur within the traveled way. If decks are not connected, differential deflections will create offsets in the riding surface that could result in potentially hazardous vehicle control problems. Maintenance of joint seals in such working joints can be difficult, hazardous to workmen, and “expensive.

Wherever the new deck joins the existing deck within the traveled way, the two should be structurally connected. However, if proper attention is not given to construction sequence and details (e.g. use of closure placement between the new deck and the existing deck), large differential deflections can cause overloading of the existing structure or distress on the new work along the joint line. The deflections may be elastic deflections resulting from the release of the false work or time-dependent deflections due to creep.

6.2.5 Differential Longitudinal Shortening

For cast in place, post-tensioned widening, it is essential that the new work be allowed to shorten longitudinally without restrain from the existing bridge. Restrains will cause some of the stressing force to be transferred into the existing bridge, creating undesirable stresses in it, and the necessary prestressing force will not be produced in the new work. When the two are to be rigidly connected in their completed state, a specific sequence of construction and the use of delayed closure placements are mandatory.

6.2.6 Vibrations from Traffic

Traffic Induced vibration has been blamed for distress occasionally observed in new construction that connects the structures carrying live loads. Research indicates that such damage is relatively rare and can be eliminated by the use of a proper construction sequence and correct design details.

6.2.7 Reclamation of Existing Deck Surfaces

Generally, bridge widening involves removal of curbs, sidewalks, or railings. This often exposes a rough surface not suitable for traffic. The original deck in these areas may have been intentionally left rough or may have been damaged during removal work, and should be restored to a smooth profile in conjunction with the widening work. This is best resolve by removing concrete to a minimum depth of 1.5 inches below grade and casting a new surface to match the adjacent grade.

It is more desirable to scarify the old deck surface and repair it as part of a combined bridge widening and rehabilitation plan.

6.3 GENERAL DESIGN CONSIDERATIONS

There are certain aspects of structure type selection, framing considerations, and design details that are unique to bridge widening. For specific design guidance, reference should be made to the AASHTO Standard Specifications for Highway Bridges or applicable design codes. However, among the questions that a designer should investigate prior to commencing design are:-

1. Can the widening be accomplished solely on the superstructure, or does the substructure also require widening?
2. If widening the substructure in necessary, was this foreseen in the original design?
3. Should the widening be balanced or unbalanced?

4. Is a parallel structure justified as alternative to widening?

In general, current design codes and loadings applicable to the route on which the structure is located should be used for bridge widening. If the original bridge was designed using outdated codes or smaller than current live loadings, designing the widening to the old codes and loadings perpetuates a deficiency. Constructing widening to current standards creates the opportunity of later replacing or strengthening all or portions of the original bridges so that the entire structure can be upgraded.

6.4 SELECTION OF STRUCTURE TYPE

Many factors influence the designer's choice of type of structure for a widening. The natural choice is to use the same type as the original, but this is not mandatory. Some of the factors that influence the choice of structure type for a widening are:

6.4.1 Aesthetic And Historic Considerations

Aesthetic and historic factors may favour maintaining the original appearances of a classical design or landmark structure. This does not necessarily mean that the same structure type should be employed. For example, the arch bridge over Nullah Lei was widened with prestressed girders outside of the truss on both sides, Multani tiles were fixed on the inner side of railing towards the deck. To all but the very astute viewer, the architectural integrity of the original design was not altered.

In most cases, however, matching the original architectural style requires the use of the same structure type. The widening should be accomplished in a manner such that the existing structure does not look "added on to." When widening a masonary arch bridge over River Soan, the original brick facing was removed, the bridge widened by redecking with side cantilevers, and the original facing reinstalled.

6.4.2 Roadway Geometric

If the widening consists of doubling the bridge width (i.e., two lanes to four lanes divided), the work is generally much less complicated and costly if the widening is done entirely on one side. The widening can be built as an independent bridge without the problems of closure placements and matching deflection characteristics. Traffic handling during construction is also simplified. When vertical clearances beneath separation structures are insufficient to allow for false work during construction of a widening the use of precast concrete or steel girders is generally required.

6.4.3 Maintenance of Traffic

The problems associated with maintaining traffic includes the safety of the public and the workmen. When a detour is not feasible and traffic must be carried through the work area, proper sequencing of construction, operations is essential to minimize these problems. It is normally preferable to do as much of the work as possible prior to the removal of the existing curb and railing. Sometimes it is possible to complete entire widening, including making the connection between old and new decks before removing the existing rails. Otherwise, temporary barriers or railings must be provided after the existing bridge railings have been removed.

The sequence of construction operations, permissible lane closure periods, minimum temporary roadway width, temporary traffic striping and signing layouts, as well as locations and details for temporary barrier railings, should all be indicated in the design documents. However, contractors should be encouraged to submit alternative schemes for approval.

High volumes of traffic may need to be carried on a bridge during its widening. When widening on both sides is performed. This situation may require that one side be completed before the other is started to minimize disruption of traffic.

When heavy volumes of commuter traffic prevent closing the existing bridge lanes except during brief off-peak periods each day, special measures may be needed. In such cases, use has been made of precast deck slabs and concrete-filled steel grating, placed during nighttime closures and either mechanically connected or connected with slot closure placement to the girders.

6.4.4 Deflection Characteristics

Deflection characteristics must be taken into account when the new deck is to be rigidly connected to the existing deck. In such cases, the designer should consider the relative deflection characteristics of the existing and the new portions of the bridge when selecting the type of structure to use in a widening.

Appreciably differences in stiffness between existing and new superstructures may cause the transfer of large portion of live load between the structures than would otherwise occur. This can result in a greater amount of live load being carried by the stiffer of the two, and transverse load distribution assumptions normally used in design may have to be modified.

For spans where differential deflection is expected to exceed quater inch, the designer should specify the sequence of attaching new work to existing. Generally a delay in the attachment of diaphragms and the placement of deck closure is needed. This is discussed in more detail in succeeding paragraphs.

6.4.5 Differential expansion characteristics

Whenever the widening is to be connected to the existing bridge, it is important the transverse deck joints be located in the superstructure of the widening in the same longitudinal locations where such joints occur in the existing bridge.

6.5 DESIGN AND CONSTRUCTION DETAILS

Standards normally used for new bridge construction should also be used for widening. These include the AASHTO Standard Specifications for Highway Bridges and the various ACI documentations listed in the Recommended References. Some construction operations that are unique to widening are discussed in this chapter. However, since each widening represents a unique situation, all of these operations do not necessary occur in every project.

6.5.2 Concrete Removal

Most bridge widening projects require that a portion of the existing bridge be removed. This is usually the railing or sidewalk and sometimes portions of the deck, substructure or wing walls. Methods of removal that could damage the existing structure should not be permitted; special care should be taken to avoid damaging any reinforcing steel that is to remain in place. The following are suggested specification provisions.

- When portions of a bridge are to be removed, the removal operations shall be performed without damage to any portion of the structure that is to remain in place.

- Existing reinforcement that is to be incorporated in the new work shall be protected from damage and shall be thoroughly cleaned of all adhering material before being embedded in new concrete.

- Before beginning concrete removal operations involving the removal of a portion of monolithic concrete element, a saw cut shall be made, taking care to avoid the reinforcing steel, to a true line along the limits of removal on all faces of the element which will be visible in the completed work.

- Reinforcement dowels exposed during the rail, curb, or sidewalk removal shall be cut off below the finished surface and the recess filled with a no shrink grout. When dowels are in a patch or overlay area, they should be cut off at the bottom of the overlay or patch.

6.6 REFINISHING EXPOSED AREAS OF THE DECK

Concrete exposed by rail, curb, or sidewalk removal may be too rough to serve as a riding surface unless a concrete or bituminous overlay is to be placed, the area must be refinished. The degree of refinishing, which can vary from minor patching to a complete leveling course, should be specified in the contractor documents.

Refinishing may consist of simply grinding off a new high spots or filling in local depressed areas with concrete repair patches. If the surface is too rough and requires extensive grinding or patching, it is generally better and more economical to mill of the entire surface to a depth of at least ¾ inch below the adjacent deck and place a contractor overlay. In either case, the recommendations in ACI 546.IR should be followed in patching or overlaying the deck surface. For example refinishing of the complete deck including concrete exposed by removal of railing and sidewalks was done by placing a three inches asphaltic layer.

6.7 TRAFFIC-INDUCED VIBRATIONS

There has been a widely held view that once concrete is placed, consolidated, and finished, it should not be disturbed until it contains sufficient initial strength. This view has led to concerns about permitting traffic on bridge decks during concrete placing operations. Traffic induced vibrations are quite noticeable to human senses, and thus understandably raise concerns among those involved in repair or widening of concrete structures. However, several reports found that vibration in bridges due to highway traffic is not as harmful as theorized. In fact, it may actually be beneficial. In situation where the reinforcing steel forms are supported by the same structural members, experience and research have shown that damage due to traffic-induced vibrations is very rare. In these cases, fresh concrete reinforcement and forms are in synchronous movement. Thus, special precautions, such as closing the bridge to traffic in such situations, are generally not necessary.

The most effective way to reduce the amplitude of traffic-induced vibrations is to maintain a smooth structure approach and deck-riding surface. Vehicle speed and weight restrictions have only a secondary effect on the magnitude of traffic-induced vibrations.

In situations where the vibrations are carried into freshly placed concrete through reinforcing steel extending from the existing bridge, damage to new concrete can occur. Such damage is avoidable in bridge by traffic control.

When a reinforcing bar moves relative to the concrete, the displaced concrete will readily flow back and forth with the bar, as long as it is still in the original plastic state. But as initial set begins, only weak, water diluted grout flows back to surround the bar. Also, cracks may develop in the plastic concrete and fill with weak material, along a horizontal plane with adjacent bars of along sloping planes running from the bar to the surface of the deck. This condition may result in a severe reduction in bond to reinforcement and premature deck spalling. Similar damage may occur in new bridge decks, if live loads from workmen or equipment are allowed directly on poorly blocked-up reinforcing steel on the outside of the construction joint bulkhead. For this reason, workers and equipment near the perimeter of a reinforced concrete deck during placing and finishing operations should be restricted to planks or runway blocked up from the forms, rather than bearing directly on any reinforcing steel that extends through bulkheads and into the concrete being placed. Although it would seem that any movement of reinforcing steel extending from a structure carrying traffic into freshly placed on a widening would result in the defects described, certain practices will generally eliminate such damage. These practices should be employed on all deck closure placements or in other situation where concrete will be placed against an existing structural elements carrying traffic, and include the following:

6.7.1 Use of Low Slump Concrete

The Michigan study, and surveys made in other states found frequent delamination in bridge decks built or widened in the 1950s and 1960s. This damage was noted in decks connecting to existing structures carrying traffic. Such damage has now been found to be related to the use of high slump (4 inch) concrete, which probably contained excess water. Similar damage was not noted in subsequent work then the slump was reduced.

Limited laboratory research at the University of Michigan also showed that high-slump concrete mixtures are especially sensitive to segregation in the plane of the reinforcing steel. In limited testing, the hydraulic pressure applied through voids cast in the plane of the top mat of reinforcement were measured. It was found that the amount of hydraulic pressure needed to rupture concrete, which was subjected to continuous vibration during its early life, was reduced from 1600 to 800 psi when the slump of the concrete was raised from 3 to 4 inch.

The maximum slump allowed during Lei bridge widening was “one to three” inches.

6.7.2 Reinforcing Details

The Texas transportation Institute found that reinforcing dowels extending straight from old concrete and lapping with the new deck reinforcing created in fresh concrete created no defects in the fresh concrete. However, they did find that, when the dowels were bent at a right angle in a horizontal plane, voids developed between the dowels a fresh concrete, although these voids were not found to cause problems in the performance of the deck. Good practice also requires that when deck closure placements are to be employed, the reinforcing bars dowels extending from the existing concrete to the new should not be connected to the reinforcing bars in the widening during concrete to the new widening but should be securely attached to reinforcing bars in the new deck just before the closure placement is made.

6.7.3 Forming Details

When deck closure placements are employed, the forms for the widening should not be connected to the existing bridge during placement of concrete for widening. However, when the closure, its supporting form should be secured to both the old and the new structure.

Differential live load deflections or relative movements between the first girder of a widening and the adjacent girder of the existing bridge cause cracking (flexure stresses) in the new deck concrete and closure problem between the two. At first consideration, it would seem that this action would also preclude in the practice of rigidly connecting decks of widening to existing bridges, which are carrying traffic. However, research done at Texas A & M University indicates that cracking of 7 3/4 inch thick concrete decks subject to dynamic loading occurred when the vertical curvature exceeded 33x100-5 arc sec/in. Field measurements of typical bridges in Georgia and Texas showed that curvatures actually produced in the deck slabs by traffic immediately adjacent to the widening doing concrete placement and thereafter were only abut one-fourth of this magnitude. These results indicate that cracking is unlikely under normal conditions.

Surveys of rigidly connected deck widening should little evidence of distress due to differential deflections caused by traffic. This is probably due to the fact that, in addition to the practices recommended previously, one or more of the following measures were taken.

1. Diaphragm between adjacent girders or a rigid temporary blocking system were used to equalize girder deflection until the deck slab gained sufficient strength Sometimes the forming system itself offers sufficient rigidity.
2. A smooth-riding surface was maintained on the deck and the approach roadway, and a good grade match was obtained where they join.
3. Traffic speed and/or allowable loads were reduced on the existing bridge during and immediately after placing new deck concrete.
4. The traffic lane adjacent to the connecting joint was closed for a few days after placing new deck concrete.
5. Temporary shoring was installed under the existing bridge during this period.

Although all of these measures have been utilized, items 3 and 4 are more economically attainable. These measures need be employed only when very flexible structures are being widened.

6.8 AVOIDANCE OF DAMAGE DUE TO DEAD LOAD DEFLECTIONS

Two important facts must be recognized when considering dead load deflection:

1. Portions of the superstructure widening must initially be built above the grade of existing structure to allow for dead load deflection, and

2. The deflected superstructure widening must meet the grade of the existing structure when the final connection is made between decks.

If proper provisions are not made to accommodate the dead load deflection, construction, maintenance, and traffic safety problems can occur.

When discussing dead load deflections, it is necessary to divide superstructures into two groups:

d. Unshored construction, such as precast prestressed concrete girders or steel girders, where the largest percentage of girder deflection occur when the deck is placed, and
e. Cast-in-place concrete superstructure construction where the deflection occur when the falsework in released.

6.8.1 Unshored Construction

It is very important to use a closure placement between the new deck and the existing bridge. For example, if the calculated deflection due to deck placement is 2 inch, at midspan and half inch calculated deflection at one-fourth of the span, will occur. When the girder has been completely loaded, the remaining 1 ¼ inch of deflection will occur.

A closure placement serves three useful purposes:-

1. It accommodated differential deflection along the joint (which is difficult to forecast accurately).
2. It provides width to make a smooth transition between the final grades; and
3. It eliminates differential deflections between the existing bridge and the widening after completion.

In precast concrete girders, the effect of creep deflection should be considered. Deflection due to creep is the result of many variables, such as the dimensions of the girder, the quality of the concrete, the concrete age, the rate of loading, etc; no single formula will suffice. ACI 435. IR, Deflections of Prestressed Concrete Members, provides a more complete discussion of this topic.

6.8.2 Cast-In-Place Concrete Construction

For cast-in-place concrete structures, the dead load deflection increases with time after the false work in release. The elastic deflection, which is only about one-fourth to one-third of the total deflection, occurs immediately after the false work is released. The remaining deflection (creep) continues at a diminishing rate, which after about four years becomes negligible. A theoretical analysis of stresses caused by the differential deflections that occur between the new and existing structures, when rigidly connected and, even when delayed closure placement is used, will usually predict that distress will occur. However, observed performance indicates that no distress occur if the procedures in this guide are followed. It is probable that plastic relaxation (creep) in the concrete allows these theoretical overstresses to dissipate before is caused to the structures.

When the total dead load deflection of the new cast-in-place structure is expected to exceed three-eighth of a inch, it is common practice to employ a closure placement after the false work is released. This is to minimize the stresses caused by differential deflections and the transfer of dead load from new to old structure. Good engineering practice suggested that the closure width and the length of the delay period after false work release and prior to placing the closure placement should relate to the amount of dead load deflection that may occur after the closure is placed.

6.8.3 Prestressed Concrete Construction

For the same span lengths, prestressed structures generally deflect less than reinforced concrete structures; thus, their use decreases the difficulty of getting a good grade match between new and existing decks. However, the use of prestressed concrete design does not eliminate the need for a closure placement. Differential longitudinal elastic shortening during stressing requires that superstructures not to be connected, until all post tensioning is complete. This longitudinal shortening continues as a result of creep. For some structures, creep may be of sufficient magnitude to warrant a greater delay in placing the closure. Accurate prediction of dead load deflection is more important for widening than for new bridges, since it is essential that the deck grades match. The total dead load deflection varies with the strength and maturity of the concrete at the time the false work is released. Thus, when determining the camber to the cast into widening for long spans, it is necessary to take into consideration the length of time the falsewrok will support the widening. This time period should be included in the contract documents. ACI 435.IR, Deflections of Prestressed Concrete Members, provides further information on the topic.

When the design calls for connecting new and existing substructures on continuous, multiple-spans, post-tensioned rigid frame structures, the piers caps, and superstructure of the existing bridge must remains unconnected to the new structure, until after the new structure is stressed. The only exceptions are piers or caps what are at the point of zero movement during stressing (e.g., the center pier of a symmetrical three-pier frame).

6.9 **CLOSURE PLACEMENT DETAILS**

The previously discussed problems created by traffic vibrations, deflections after release of false work, and longitudinal shortening due to post-tensioning can be mitigated by the use of longitudinal expansion joints. However, when the junction of the widening to the existing deck falls within the traveled way, longitudinal expansion joints are generally avoided because such joints create maintenance problems.

Closure placements on short spans or on very narrow widening may not be needed if deck concrete is placed fast enough to permit dead load to deflection and completed prior to final strike off and initial set of the concrete. Retarding additives can be utilized to ensure deflection is completed before initial set of the concrete.

When closure placements between the new deck and the existing deck are used, the construction sequence and details employed are critical to the successful performance of the structure. Their degree of importance varies from bridge to bridge due to the many variables involved. The recommended width of closure placements is approximately 18 in.

The use of a closure placement accomplishes two purposes;

- It permits the widening to remain isolated from live load deflections and vibrations from Traffic on the existing bridge, and.
- It allows dead load deflection and prestressing shortening of the widening to reach a stage where the portion of the new deck that connects to the old will not be overstressed due to differential movements between old and new structures.

When deck closure placements are employed, diaphragms connecting new to old girders are left disconnected until all other work is completed, except for the placement of the closure. Such diaphragms should be connected just prior to placements of the closure.

6.9.1 Attachment to Existing Bridge

Structures with large deck overhangs should have a sufficient width of concrete removed from the overhang to permit lap splicing of the original transverse deck reinforment with that of the widening. Structures with small or no overhangs should either be connected to the widening with dowels or have sufficient transverse reinforcement exposed to permit splicing by welding or mechanical connections.

Cutting a seat into the existing explorer giving as a means of support has proven to be unsatisfactory because of difficultly in reinforcing area around the seat. Double-row patterns of dowels perform better than a single row. Dowels may be grouted or epoxied into the existing concrete. Presized encapsulated resin cartridges have proven to be a fast, reliable, and economical method to set dowels.

The preferred method is grouting, where neat cement paste is specified for the bonding agent. This method requires a hole, sloping one vertical to three horizontal or steeper so the fluid grout could not escape. Nonshrink grout performs better than other Portland cement grouts for this use. Some epoxies tend to creep more than concrete under sustained loads; therefore, these materials should be used only in connection, which are subjected solely to live or impact loads.

The holes should be drilled by methods that do not shatter or damage the concrete adjacent to the holes. They should be located at least 3 inches from the edge of the concrete and be no more than quarter inch larger than the diameter of the dowels or as recommended by the manufacturer. The holes must be free of dust and drilling slurry and in surface dry condition before placing the grout or epoxy. The holes are then filled with grout or epoxy before the dowels are inserted. As an alternative to inclined holes, horizontal holes about ¾ inch larger than the dowels may be drilled and the dowels bonded in place with nonshrink grout. The dowel is centered in the hole and the grout is then injected into the hole so that filling is accomplished outward from the base of the hole. A gasket is used around the dowel at the face of the hole to retain the grout while allowing the air to escape.

6.9.2 Reinforcement

During placement of deck concrete in the widening reinforcing bars protruding from the new deck into the closure space should be kept completely free of contact with the existing reinforcing steel, concrete forms or attachments thereto.

During placement of deck closure concrete, the new and existing transverse reinforcing steel within the closure should be securely connected together or to common longitudinal reinforcement. Reinforcing bars extending from the existing deck should be straight rather than hooked. Reinforcing bars extending from the existing deck that are too short to give sufficient length may be extended by approved mechanical connections or full-strength welds. Welding may be used when the extension being welded is free from restraint during the welding process to permit shortening of the bar as the weld cools.

The American Welding Society's DI.479, Structural Welding Code for Reinforcing steel, contains recommended details for making welded splices in reinforcing steel. This code also suggests making a chemical analysis of the steel to determine its weld ability and gives procedure for welding splices if chemical composition is unknown.

Longitudinal reinforcing bars should be placed in the closure placement to distribute shrinkage cracks and minimize their width.

6.9.3 Forms

Forms for the deck closure placement should be supported from the superstructure on both sides of the closure. They then act as an articulated ramp to spread the effect of any different vertical movements over the widths of the closure. The forms should not be placed between old and new structures until the placement of all other concrete in the widening has been completed and the false work released.

6.9.4 Concrete

Specific requirements for concrete are necessary for encasing reinforcing steel that is subject to vibration from external forces during the first few days after placement. This applies to concrete for closure placements, and deck widening when closure placements are not used and traffic are allowed on the old bridge during construction. Minimum cement content of 564 lb/yd3, a maximum of 0.45 water cement ratio, and a maximum slump range of 1 to 3 in should be employed. High early strength cement, no shrink types of cement, chloride free proprietary high early strength cements are sometimes used.

6.9.5 Time of placements

The time of placement of concrete for closure placements depends on the type of structure.

1. For steel girders or precast concrete girder bridges, closure placements may be made as soon as the full dead load is on the widening. For widening consisting of more than one girder, the exterior railing need not be placed prior to closure.

2. For cast-in-place concrete construction, a delay after removal of false work should be provided to allow the relatively early plastic dead load deflection to occur before the decks are connected. The length of the delay period, alongwith the width of the closure placement, should be engineered to accommodate the dead load deflection that it will occur in the widening after the closure in placed. The deflection is primarily the result of creep in the concrete superstructure.

In concrete, the amount of creep (deflection) is a direct function of the level of stress in the concrete. The rate of creep decrease with the age of the concrete and the length of time since the formwork was removed. Any such deflection of the girder in the widening which occurs after the deck closure placement has been made will produce differential displacement induced stresses in the closure and adjacent deck. However, these stresses, in turn, will be reduced by creep in the closure concrete. Thus, the rate of deflection of the girders must be permitted to decrease to a level, which can be tolerated by the closure concrete before closure concrete is placed.

The combined actions, with younger concrete in the closure in the widening, make the determination of the delay period very difficult to calculate. Therefore, the delay period is normally based on experience.

The California Department of Transportation (CLATRANS) has required for several years that whenever the false work is released at the earliest permitted date, the closure placement shall not be placed until at least 60 days after the false work is released. As an alternative, if the false work is left in place for at least 28 days, then the closure concrete can be placed after 14 days false work release.

A somewhat empirical, but more rational, approach is now under consideration for use by CALTRANS. This approach relates the required delay to a combination of the expected long-term deflection at midspan of the new structure (widening) and the maturity of the new concrete at the time of false work removal.

The minimum delay in days after false work release and prior to placing deck closure concrete equals the delay factor shown in the following table; minus twice the number of days the completed span (other than railing) is supported on false work (age of concrete). However, in no case shall this delay be less than 1 day for each 0.01 ft of deflection.

Delay factor table

Calculated ultimate midspan Deflection foot	Delay factor days
0.05	20
0.10	32
0.20	51
0.30	66
0.40	80
0.50	94

For example, if the ultimate deflection of a span is exceeded to be 0.30 foot and the false work was released 15 days after the last concrete was placed, then the deck closure can be made 36 days or later after the false work was released [6=66-(2x15)].

This procedure is reasonable for 18-inch wide closure placement in 7 to 8-inch. decks. Some reduction could be made for closure placements of greater width, while for licker deck sections, an increase in delay time would be justified.

6.10 SUBSTRUCTURE DETAILS

Typically, the new substructure is rigidly attached to the existing substructure. In unusual cases, closure placements in footing and substructures have been used to prevent transfer of load from new to old structures.

6.11 SUMMARY

There are many problems that are unique to the widening of bridges. Most of the major problems can be avoided by decisions regarding the choice of structure type, whether or not to connect the deck of the widening to the deck of the existing bridge, and the method and sequence of making such connections.

The joint between the widening and the existing bridge generally occurs within the area of the deck that will be traversed by vehicles. It is recommended that the decks be structurally connected. The type of bridge used for the widening does not have to be the same as the existing. Economy, site geometric (e.g. roadway or water way clearance), and aesthetics should determine the choice. However, if the two spans are to be structurally connected, the live load deflection characteristics of the type chosen for the widening should be similar to those of the existing bridge.

When the deck of a widening is to be structurally connecting to the existing deck, it is generally recommended that the final connection be delayed until the widening is nearly complete. This will avoid the possibility of the new work caused by

4. Vibrations of traffic on the existing bridge,
5. Dead load deflection of the widening that occur as deck concrete is placed or as false work is removed, and
6. Shortening of the new work if it is longitudinally post-tensioned.

For continuous post-tensioned spans of rigid frame bridges, the new substructures should also not be connected to the existing bridge in any way that would prevent the longitudinally shortening of the new work. For cast in place spans, it may be advisable to delay making this final connection for several days after the false work is released to allow for some of the more rapid early dead load deflection caused by creep in the concrete to occur (see section for suggested delay times).

The recommended method of making structural connections between widening and existing deck is to leave an approximately 18-in. gap between the two which is later filled with high-quality (0.45 maximum w/c), low-sump concrete. This closure placement should be reinforced with top and bottom mats of reinforcing bars which extending out of both and new extending deck slabs. Such reinforcement must all be securely tied together to minimize differential movements and the resulting damage to the fresh closure concrete caused by vibrations from traffic. However, such reinforcing steel ties should not be made until just before the closure concrete is placed. Likewise, the connection of diaphragms between the existing bridge and the widening, and the installation of the forms for the closure placement, should not be done until just before the closure is placed