# 10.00 Waterway crossings

## 10.01 Bridge crossings

### (a) General

Bridges are generally the preferred means of crossing an open channel or urban waterway, particularly in the following circumstances:

(i) where the road elevation is well-above the stream's bed level;

(ii) fish passage is required along the waterway for a threatened fish species (refer to Table 9.07.1);

(iii) there is a high degree of environmental sensitivity associated with the waterway and/or its banks.

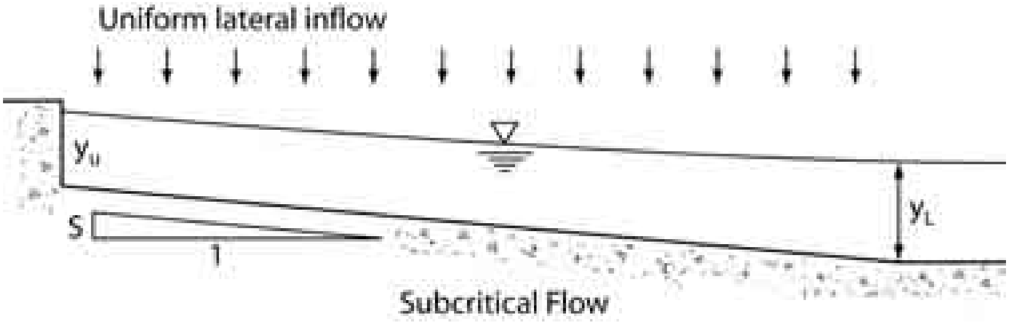
The design of a bridge is a complex matter requiring input from a multi-disciplinary team including suitably qualified engineers. Design guidelines may be obtained from Department of Main Roads (2002 & 2000a) and AustRoads (1994, 2005). Guidelines on scour control around bridge structures are provided in AustRoads (1994, 2005) and Appendix C of Witheridge (2002). The impact of a bridge and its approaches on flood levels in major/extreme events may also need to be assessed through specialist floodplain modelling.

### (b) Hydraulics of scupper pipe outflow channels

Roadways represent a major source of stormwater pollution. Stormwater runoff from bridges should be collected and filtered through riparian vegetation and/or other appropriate treatment measures to ensure compliance with water quality objectives before being released into the waterway.

Stormwater runoff is typically collected by scupper pipes and discharged into a "side flow channel" or stormwater pipe attached to the bridge deck. Backwater analysis of a *side flow channel* may be based on the hydraulics of a lateral spillway channel.

A *lateral spillway channel* is an open channel which receives lateral inflow along its length. Benefield et.al. (1984) describes the hydraulics of three channel flow conditions as shown in Figures 10.01 to 10.03.



**Subcritical flow with subcritical tailwater**
**Figure 10.01**

For a rectangular channel with downstream subcritical flow depth ($y_L$), the upstream water depth ($y_u$) may be estimated using Equation 10.01.

$$y_U = \left[\frac{2(y_C)^3}{y_L} + \left(y_L - \frac{S.L}{3}\right)^2\right]^{1/2} - \frac{2.S.L}{3} \qquad (10.01)$$

where:

$y_u$ = upstream water depth [m]

$y_c$ = effective critical water depth at end of channel where total flow, $Q = q.L$

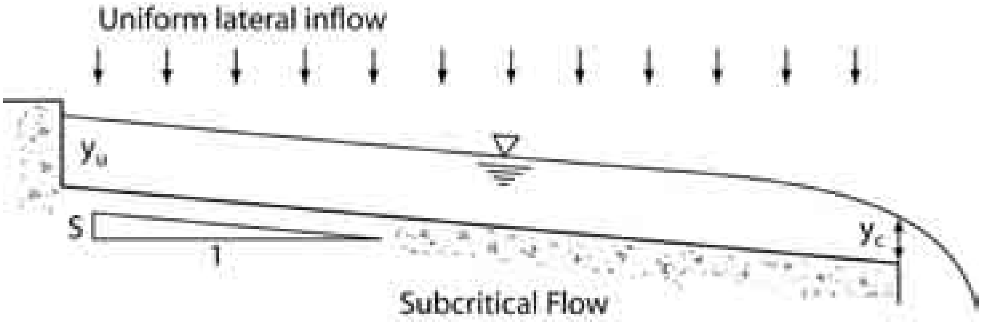
$Q$ = total flow rate [m$^3$/s]

$q$ = lateral inflow rate per unit length [m$^3$/s/m]

$L$ = length of channel over which lateral inflow occurs [m]

$y_L$ = flow depth at downstream end [m]

$S$ = channel slope [m/m]

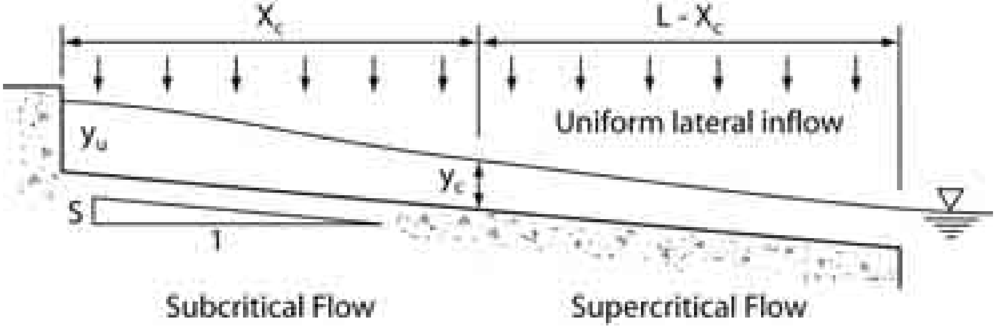


**Subcritical flow with critical depth at tailwater**
**Figure 10.02**

For a rectangular channel with critical depth ($y_c$) at the downstream end of lateral inflow, the upstream water depth ($y_u$) may be estimated using Equation 10.02.

$$y_U = \left[ 2(y_C)^2 + \left( y_C - \frac{S.L}{3} \right)^2 \right]^{1/2} - \frac{2.S.L}{3} \qquad (10.02)$$

For zero channel slope ($S = 0$): $y_u = 1.73\, y_c$



**Combined subcritical and supercritical flow**
**Figure 10.03**

For a trapezoidal channel with critical depth occurring within the length of the channel, the location of critical depth may be determined from Equation 10.03.

$$y_U = \left[ 2(y_C)^2 + \left( y_C - \frac{S.X_C}{3} \right)^2 \right]^{1/2} - \frac{2.S.X_C}{3} \qquad (10.03)$$

where:

$X_c$ = length of channel containing subcritical flow [m]

$$X_C = \left[ \frac{g.A^3}{q^2.B} \right]^{1/2} \qquad (10.04)$$

where:

$g$ = acceleration due to gravity [m/s$^2$]

$A$ = cross sectional area of trapezoidal channel [m$^2$]

$B$ = base width of trapezoidal channel [m]

Equations 10.01, 10.02 and 10.03 will all slightly underestimate the upstream flow depth because they ignore friction loss. The degree of underestimation will depend on the roughness of the channel.

## 10.02 Causeway crossings

A causeway overtopped by minor flood flows will act as a broad-crested weir with the discharge principally being controlled by the tailwater conditions. During low flows, upstream water levels may be independent of downstream conditions. Submerged flow conditions typically occur when tailwater levels are high—during which flow passing over the causeway remains subcritical. Book 7 of ARR (1998) and Department of Main Roads (2002) provide details of design methods for both tailwater situations.

An additional consideration in the design of causeways is the safety of vehicles and pedestrians when a causeway is overtopped during flooding. A maximum depth*velocity product (d*V) of 0.4, a maximum flow depth of 200mm and a maximum energy level of 300mm should apply to trafficable flow conditions for urban causeways crossed by two-wheel driven vehicles (refer to Table 7.04.1). Warning signs should clearly indicate likely trafficable hazards. These warning signs should indicate that safety risks exist whenever water is passing over the causeway.

Fish passage conditions are improved if the profile of the causeway follows the *natural* cross section of the streambed, thus providing variable flow depths over the causeway; however, such conditions are generally <u>not</u> recommended for reasons of traffic safety. In general, the use of causeways is not recommended in fish passage streams.

## 10.03 Ford crossings

Ford crossings should only be used for very low traffic volumes, or for the crossing of "dry" alluvial stream beds (i.e. when the risk of causing water contamination is minimal).

Ford crossings of clay-bed streams need to be suitably stabilised to minimise damage to the stream bed even if flow is not occurring within the channel while the crossing is being used. Crossings protected with rock (say greater than 150mm) are likely to require a downstream sill, such as a log, to minimise displacement of the rock. Both the rock and sill should be recessed into the bed to form a surface level with the natural streambed.

Fixed bed ford crossings (e.g. concrete bed level crossings) should not be used to cross alluvial streams (i.e. sand or gravel-based streams) if fish passage conditions are to be maintained within the waterway. If a fixed bed ford crossing must be used in alluvial streams, then their performance and impact on fish passage must be regularly monitored.

## 10.04 Culvert crossings

Designers are referred to the design procedures prepared by the Department of Main Roads (2002).

### 10.04.1 Choice of design storm

Table 7.02.1 provides recommendations for the selection of design storms for road culverts.

Some local governments may require flood free access to new residential development during the major design storm to provide safe passage for emergency vehicles. As a result, some culverts will be designed to carry the major design storm. In such circumstances, consideration should be given to the impact of flows greater than the major design storm as discussed in Sections 7.03.2, 9.03.2 and 10.04.7 of this Manual. The potential impacts of full or partial debris blockage of the culvert must also be considered as discussed in Section 10.04.10.

If a local government specifies a design storm less than the 1 in 100 year ARI, it would be considered reasonable for the local government to also require that the effects of a 1 in 100 year ARI design storm shall not unreasonably:

(i) increase the flooding of critical areas defined by the local government, such as habitable floor levels;
(ii) adversely affect the "value" or "use" of adjacent land;
(iii) cause unacceptable property damage.

### 10.04.2 Location and alignment of culverts

The location of a roadway crossing is usually governed by the location of an existing road reserve, but when circumstances allow, waterway crossings should ideally be located:

(i) on a straight section of the waterway;
(ii) well downstream of sharp channel bends;
(iii) on a stable channel section;
(iv) upstream of a channel riffle (i.e. locating the culvert within a "pool" if a pool-riffle system exists within the stream).

Wherever practical, culverts should be aligned with the stream channel; however this can significantly increase the length and cost of skewed culverts compared to a culvert aligned perpendicular to the roadway. It is generally not considered acceptable to realign an existing waterway channel simply to reduce the length of a culvert. The advantages and disadvantages of each option should be considered on a case-by-case basis.

If it is not practical to align a culvert barrel with both the upstream and downstream channels, then priority should be given to aligning the culvert outlet with the direction of the downstream channel.

## 10.04.3 Allowable afflux

In choosing the allowable afflux caused by the culvert, designers shall consider the following:

(i) the afflux must not cause unacceptable damage to adjacent properties, or adversely affect the use of the land;

(ii) adequate freeboard (minimum desirable 100mm) should exist between the design flood water surface and the lowest part of the road cross section at the crossing.

## 10.04.4 Culvert sizing considerations

When sizing a culvert, the following recommendations/issues should be considered:

(i) The general minimum size of all cross drainage culverts should be 375 mm diameter for pipes and 375mm height for box culverts.

(ii) The larger the culvert cells, the lower the risk of debris blockage.

(iii) To minimise the effects of debris blockage, and to minimise the risk of a person (being swept through the culvert) drowning, all reasonable and practical measures should be taken to maximise the height of the culvert, even if this results in the culvert's hydraulic capacity exceeding the design standard.

(iv) The Department of Main Roads (2000b) provides guidelines on minimum culvert sizes for terrestrial fauna passage.

(v) If fish passage through the culvert is considered necessary, then the minimum flow area may be controlled by fish passage requirements as discussed in Section 10.04.14.

(vi) In multi-cell culverts it may be desirable to include one or more larger cells. These cells are usually recessed into the channel bed and are designed to allow limited sedimentation to occur within the culvert to simulate *natural* streambed conditions to aid fish passage.

## 10.04.5 Preliminary sizing of culverts

A first estimate of the culvert size may be obtained using Equation 10.05 (culverts flowing full only).

$$\Delta H = C \,.\, (V^2/2g) \qquad (10.05)$$

where:

$\Delta H$ = approximate head loss through culvert flowing full (i.e. outlet control) (m)

$C$ = constant equal to 1.5 for large culverts, or 1.7 for small, high-friction culverts

$V$ = average flow velocity within culvert = Q/A (m/s)

$Q$ = total flow rate passing through culvert ($m^3/s$)

$A$ = total flow area of culvert ($m^2$)

$g$ = acceleration due to gravity ($m/s^2$)

Equation 10.05 may be rearranged and presented as Equation 10.06.

$$A = Q/(3.6\Delta H^{0.5}) \qquad (10.06)$$

Following preliminary design, a more refined culvert size may be obtained from Equation 10.07 (culverts flowing full only).

$$\Delta H = \left( K_e + \left( \frac{2g\,Ln^2}{R^{4/3}} \right) + K_{exit} \right) . \left( \frac{V^2}{2g} \right) \qquad (10.07)$$

where:

$K_e$ = entrance loss coefficient (assume 0.5 if unknown)

$L$ = length of culvert (m)

$n$ = average Manning's roughness of culvert

$R$ = hydraulic radius of culvert flowing full (m)

$K_{exit}$ = exit loss coefficient (assume 0.8 if unknown) otherwise the exit loss component equals the change in velocity head from within the culvert ($V^2/2g$) to a downstream location where the flow has expanded to approximately full channel width ($V_{d/s}^2/2g$), thus:

$$K_{exit}(V^2/2g) = (V^2/2g) - (V_{d/s}^2/2g) \qquad (10.08)$$

### 10.04.6 Hydraulic analysis of culvert

Final hydraulic analysis of a culvert should be carried out either using hand calculations (minor culvert only), or using numerical modelling. Culverts should not be sized using *Inlet Control Charts* if the outlet of the culvert is likely to be drowned (i.e. when *Outlet Control* conditions exists). It is noted that *Inlet Control* conditions require free surface flow conditions to exist at the culvert outlet.

### 10.04.7 Consideration of flows in excess of the design storm

The likely effects of channel flows corresponding to a storm event in excess of the design ARI storm event used in the culvert design should be considered and the consequences discussed with the local government (also refer to Sections 7.03.2 and 9.03.2 of this Manual).

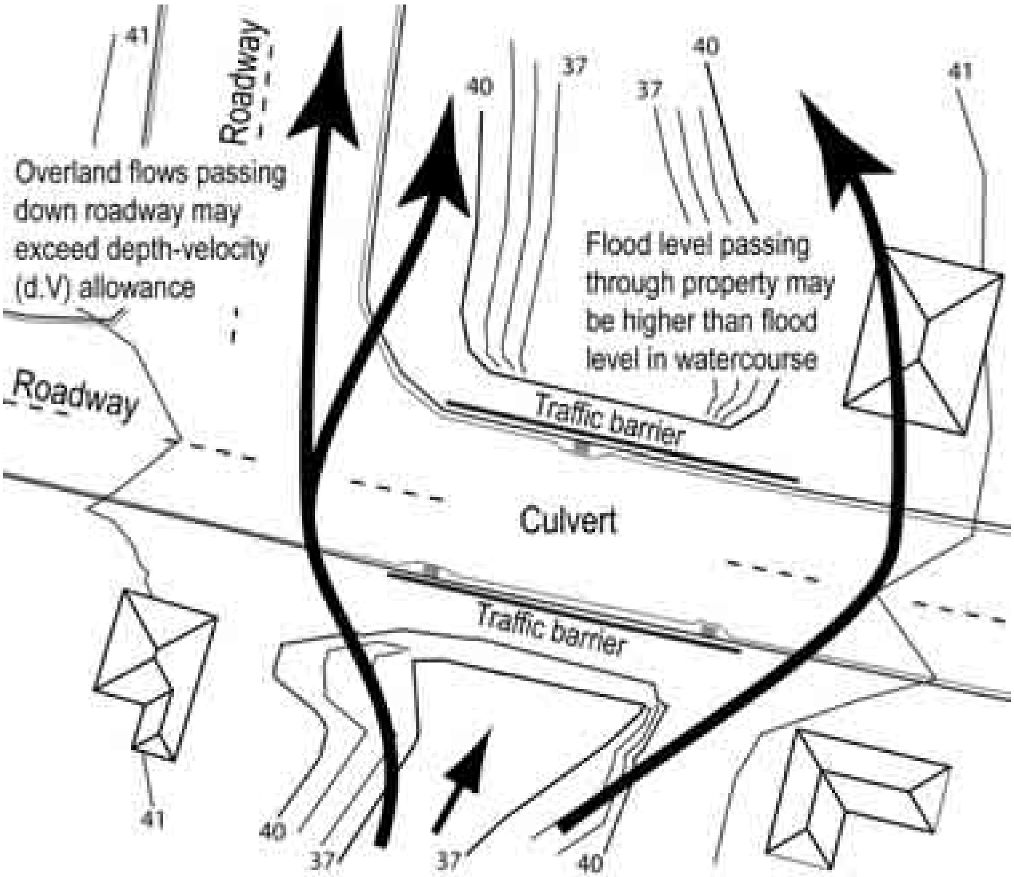
Consideration of the potential impact of large flows need to include the following:

(i) Whether these impacts can be adequately predicted or modelled.

(ii) The likelihood of significant debris blockage of the culvert, roadway fences and crash barriers.

(iii) The relative elevation of property floor levels (residential or commercial) upstream and adjacent to the culvert.

(iv) The path of overflows (e.g. overflows may pass through downstream properties before entering the downstream channel) Figure 10.04.



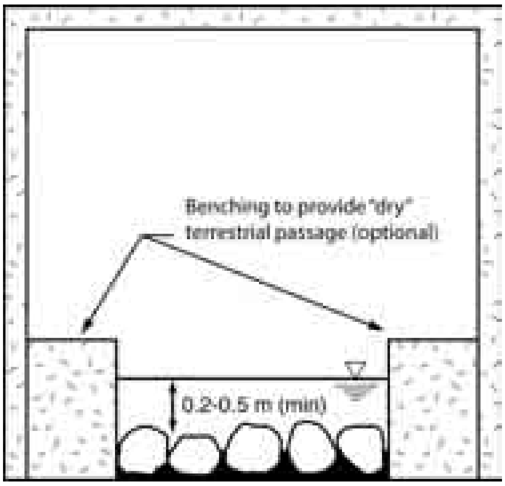
**Example flow path of overtopping flows**
**Figure 10.04**

## 10.04.8 Culvert elevation and gradient

Generally the culvert's invert should follow the stream's natural gradient.

The invert of fish-friendly culverts should be set at an elevation that allows at least 0.2 to 0.5 m flow depth during periods of extended low flows (i.e. base flow conditions in perennial streams, or flow conditions following prolonged wet weather in ephemeral streams).

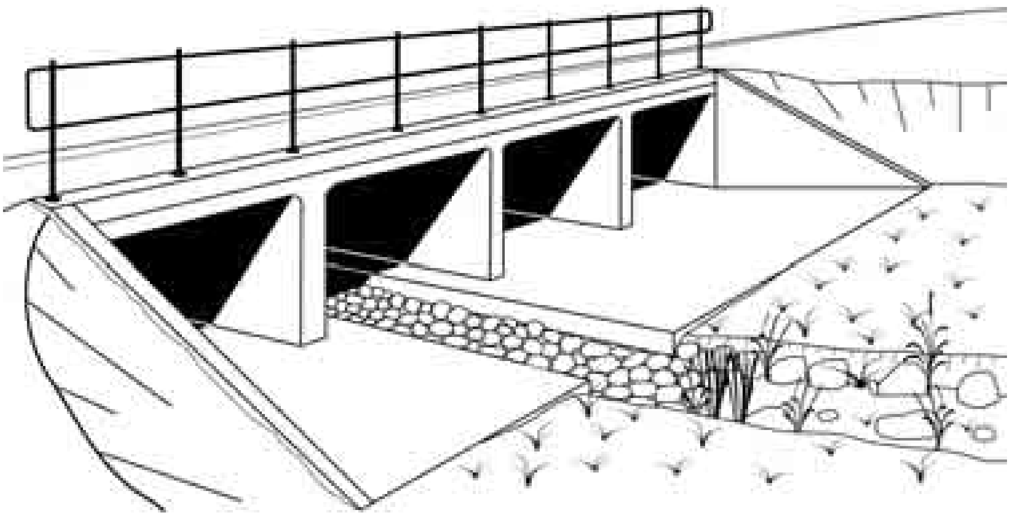
If rock or natural bed material is allowed to settle along the base of the culvert, then an allowance must also be made for the expected depth of this imported bed material (Figure 10.05). This may require the culvert to be set significantly more than 0.2 to 0.5 metres below the existing bed level.



**Minimum desirable flow depth over placed or settled bed material**

**Figure 10.05**

In multi-cell culverts, at least one cell should be recessed into the bed to form a “wet” cell. The remaining cells can be elevated as “dry” cells suitable for terrestrial passage (Figure 10.06). An appropriate adjustment to the flow area and roughness needs to be made in the hydraulic analysis. Table 9.03.3 provides information on Manning’s roughness values for rock-lined culvert cells.



**Multi-cell culvert with wet and dry cells**

**Figure 10.06**

Drop inlets shall not be used on culverts that are required to be fish friendly, unless suitable fish passage conditions are provided. Fish friendly drop inlets usually consist of a pool-riffle system, or a rock chute with maximum 20:1 to 30:1 gradient.

## 10.04.9 Minimum cover

Depending on the concrete/loading class, the generally accepted minimum allowable fill is 300mm over Concrete Pipes, 100mm over Reinforced Concrete and Slab Link Box Culverts (RCBC and SLBC respectively) and Reinforced Concrete Slab Deck Culvert (RCSDC) and 600mm over Corrugated Metal Pipes.

Designers should refer to the latest recommendations from the Concrete Pipe Association of Australasia to confirm desirable minimum cover requirements.

## 10.04.10 Debris deflector walls

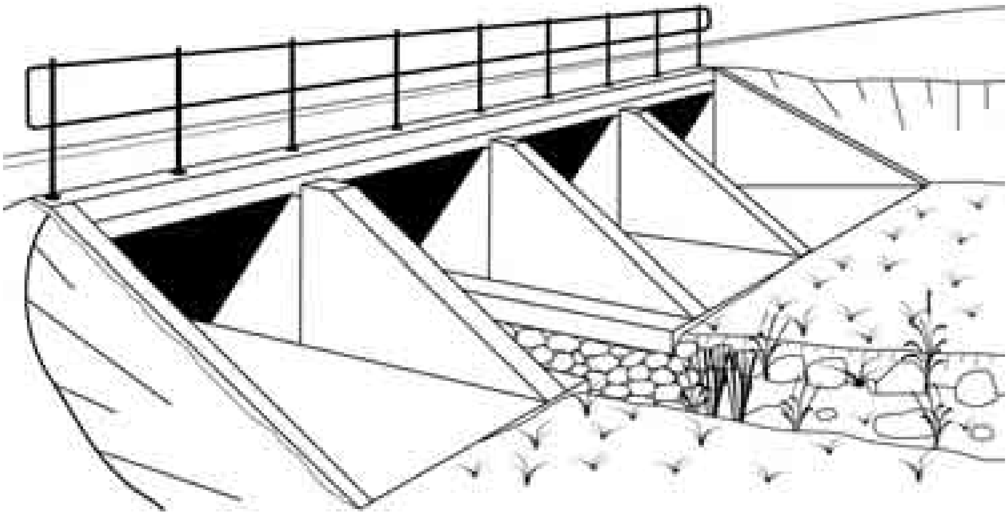
Waterway culverts can experience debris problems in a number of circumstances, especially where:

(i) the upstream waterway is heavily vegetated and is currently undergoing channel expansion due to changing catchment hydrology;
(ii) the waterway has a large catchment area;
(iii) the culvert has insufficient clear waterway area to allow the free passage of debris;
(iv) the culvert is downstream of potential slip areas that could result in significant debris flow;
(v) the culvert has a history of debris problems.

Hydraulic analysis of a culvert should take reasonable consideration of likely debris blockage. Some local governments adopt 100% blockage of all solid railings and traffic barriers, but no blockage of the culvert cells. In such cases, consideration of possible debris blockage of the culvert cells should be considered when designing 1 in 100 year ARI *flood free* culverts (i.e. those culverts where overtopping flows are not desirable during a 1 in 100 year ARI event).

The typical debris blockage allowance is 10% to 20% of the culvert flow area depending on the extent of vegetated waterways upstream of the culvert.

One means of maintaining the hydraulic capacity of culverts in high debris streams is to construct debris deflector walls (1V:2H) as shown in Figure 10.07. The purpose of these walls is to allow the debris raft to rise with the flood, thus maintaining a relatively clear flow path under the debris.

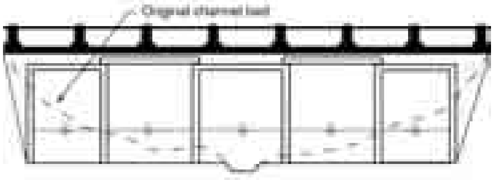


**Culvert inlet with debris deflector walls**

**Figure 10.07**

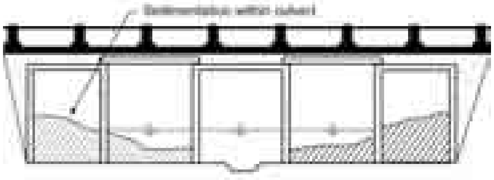
## 10.04.11 Sediment control measures

Multi-cell culverts typically experience sedimentation problems within the outer cells of the culvert. This is primarily caused by the stream channel trying to reform the *natural* channel cross section that existed prior to construction of the culvert as shown in Figure 10.08.



**Multi-cell culvert showing original channel cross section**

**Figure 10.08 (a)**



**Typical long-term sedimentation within alluvial waterways**

**Figure 10.08 (b)**

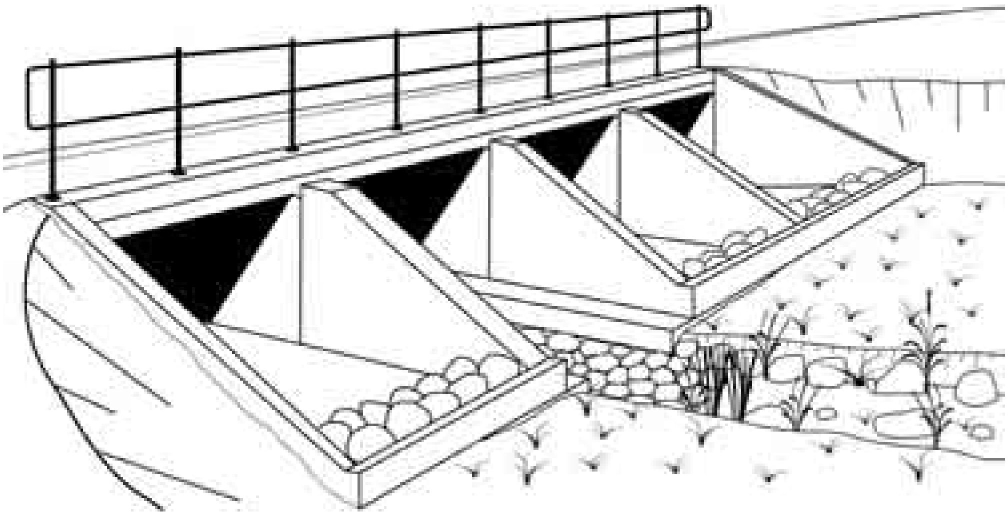
Sedimentation of culverts can be managed using one or more of the following activities:

(i) formation of an instream sedimentation pond upstream of the culvert;

(ii) formation of a multi-cell culvert with variable invert levels such that the profile of the base slab simulates the *natural* cross section of the channel;

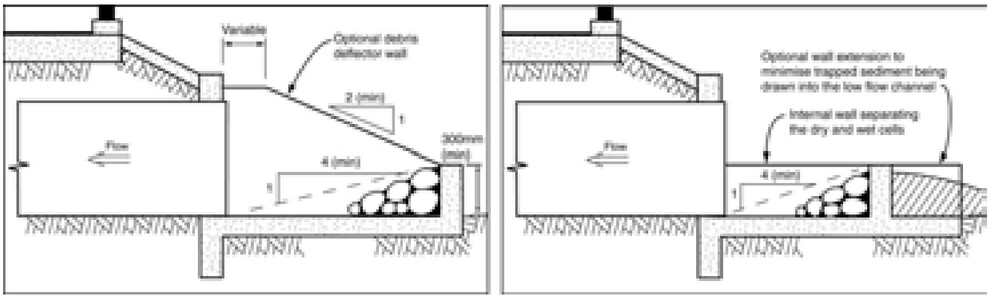
(iii) installation of *Sediment Training Walls* on the culvert inlet.

Sediment training walls reduce the risk of sedimentation of the outer cells by restricting minor flows to just one or two cells as shown in Figures 10.09 and 10.10.



**Sediment training walls incorporated with debris deflector walls**

**Figure 10.09**



**Various arrangements of sediment training walls with (left) and without (right) a debris deflector wall**

**Figure 10.10**

If the culvert is located within a terrestrial passage corridor, it may be necessary for grouted rock ramps (Figure 10.10) to be formed on the downstream face of the training walls to assist in the passage of terrestrial wildlife such as tortoises. As with all aspects of sediment training walls, the application of this feature should be assessed on a case-by-case basis.

As with debris deflector walls, the use of sediment training walls should be restricted to those culverts where the benefits gained by their use outweigh the additional costs. In most cases, their use will be restricted to clay-based creek systems.

It should also be noted that the design of sediment training walls is still in the early stages of development and further refinements are likely to occur in the future.

### 10.04.12 Roadway barriers

Prior to the installation of any traffic safety barriers, consideration must be given to their impact on flood levels and terrestrial passage.

During overtopping flows, raised median strips can raise upstream flood levels as well as restrict traffic movement to one side of the road. In critical flood control areas, it may be necessary to use a painted median.

### 10.04.13 Terrestrial passage requirements

Terrestrial passage is normally required to be incorporated into the design of a culvert when the road crosses a fauna corridor and traffic conditions on the road are such that unacceptable road kills are likely occur.

Dry passage should extend through the culvert along one or both sides of waterway channel as required. These dry paths should extend along the wing walls until they intersect with the waterway bank.

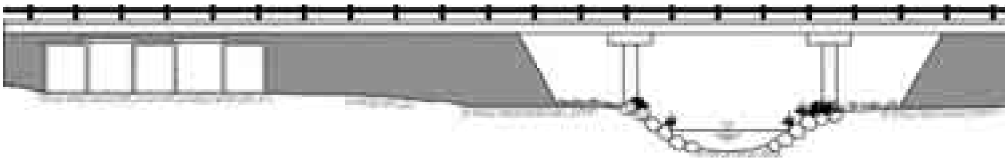
Guidelines on the integration of terrestrial passage into waterway crossings may be obtained from Department of Main Roads (2000b).

### 10.04.14 Fish passage requirements

Fish passage consideration is normally required in the following circumstances:

(i) as directed by DPI Fisheries or the local government;
(ii) when identified within a Wildlife Corridor Map;
(iii) streams containing permanent water (pooled or flowing);
(iv) streams containing aquatic life that requires passage.

Fish passage may also be required through dry-bed culverts located within a floodplain adjacent to a bridge crossing (Figure 10.11). Such conditions would normally exist within river systems containing fish species that primarily migrate along floodplains during high flows.



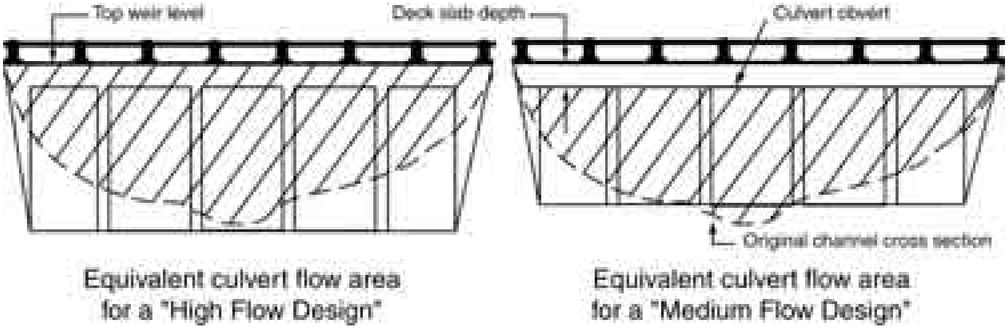
**Floodplain culvert adjacent a bridge crossing**

**Figure 10.11**

Desirable hydraulic conditions for fish passage may exist for a wide range of flow rates. Witheridge (2002) describes three design conditions: *High*, *Medium* and *Low Flow Designs*. Ideally a fish-friendly culvert should be based on a *High Flow Design*, however where such a design is not economically practical, then consideration may then be given to a *Medium Flow Design*, or a *Low Flow Design*.

A *High Flow Design* requires a minimum culvert flow area equal to the natural or existing channel flow area below the culvert's top "weir" elevation (Figure 10.12).

A *Medium Flow Design* requires a minimum flow area equal to the natural or existing channel flow area below the culvert's obvert (Figure 10.12). In addition, all reasonable efforts should be taken to minimise the depth of the deck slab, thus minimising the cross sectional area of the deck.



Equivalent culvert flow area for a "High Flow Design"

Equivalent culvert flow area for a "Medium Flow Design"

**High and medium flow area requirements for fish friendly culverts**

**Figure 10.12**

A *Low Flow Design* requires suitable fish passage conditions when flow depths are in the range of 0.2 to 0.5 metres. No minimum culvert flow area is specified.

If a riffle system, or pool-riffle systems is established through the "wet" cell, then resting pools should ideally be established at the inlet and outlet of the culvert.

Guidelines for the design of waterway crossings sympathetic to aquatic and terrestrial passage are provided in Witheridge (2002). Also refer to Sections 9.07.3 and 9.07.4 of this Manual.

## 10.04.15 Outlet scour control

Discussion on the attributes of various energy dissipaters is provided in Section 8.06 of this Manual. In most cases, safety concerns will prevent the use of most plunge pool and impact energy dissipaters, thus limiting downstream scour control to the use of outlet rock pads.

The required depth of apron cut-off walls is dependent on a number of factors including flow rate, outlet velocity, and type of bed material. A minimum depth of cut-off wall penetration of 0.6 metres is recommended unless otherwise directed by the local authority. In critical situations, designers should consult Chiu & Rahmann (1980) and Peterka (1984) for procedures concerning the determination of required cut-off wall depths.

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