ice in cold regions. It is therefore beneficial in such regions to cover the soil with rice straw mat after seeding to prevent the aforementioned phenomena. This method is often partially executed in consideration of the dried condition of the slope along with other forms of covering work. It is necessary to secure the mats with ropes and spit.

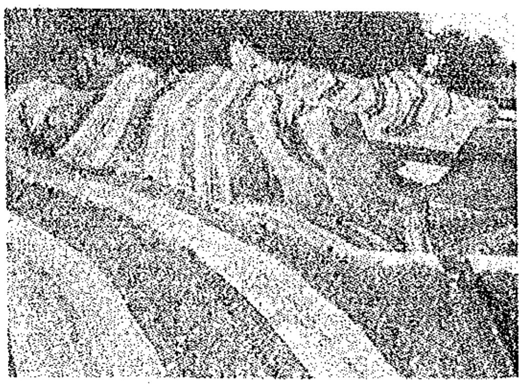


Diagram 3-22

4) Net Covering Work

This method is employed in places where the gradient is excessive and when materials cannot be maintained and secured with other forms of covering work. This work may be segregated into those which are executed in replantable areas and those which are executed in areas where replanting is difficult.

The method employed in replantable areas are accomplished by securing nets applied with seeds, fertilizers, and other organic mediums in order to achieve both seeding and covering work effects simultaneously. The method in areas where replanting is difficult is accomplished simply by covering the overall ground with nets to physically prevent rocks and surface soil from falling. Although surface soil is produced over a prolonged period of time on such surfaces allowing entry of vegetation upon stabilization, plant seeds must be sowed to enhance this effect.

4. Slope Seeding Work

Overall vegetative covering is necessary in order to permanently stabilize the hillsides. The hillside slope is left bare between the steps and planting trees for some time after completion with methods other than the covering work. Early stage overall replanting using trees and weeds must therefore be considered since these bare surfaces are highly erodible by rainfall, wind, freezing during this period. Slope seeding works therefore, is a method to sow seeds throughout the slope directly individually or in combination with other works to vegetate the ground with the germination and growth of vegetation. Methods such as sowing seed and fertilizer mixture into ditches and covering with soil, or by overall sowing on hillsides which are then covered with straw and straw mats are currently employed. Additionally, seed and fertilizer mixture are being sprayed on slopes with pump, and then adhesives such as asphalt emulsion sprayed over in order to prevent washouts of sprayed seed and fertilizer. Furthermore, seeding is accomplished by helicopters in distant areas where material transportation is difficult, as well as when large devastated areas must be replanted in a short period of time. The seeds of trees and grasses mixture are comprised of seeds with different characteristics, and the blending proportions are adjusted in accordance with the germination periods as well as initial growth conditions. Additionally, the seed quantity is increased or decreased in accordance with the number of individual growth per area unit.

Characteristics of trees and grasses for slope Seeding Work

1) Germination and Initial Growth

Grasses employed for slope seeding work must germinate simultaneously within a short period of time from seeding and exhibit rapid initial growth in order to cover the unstable surface soon after the completion of hillside work.

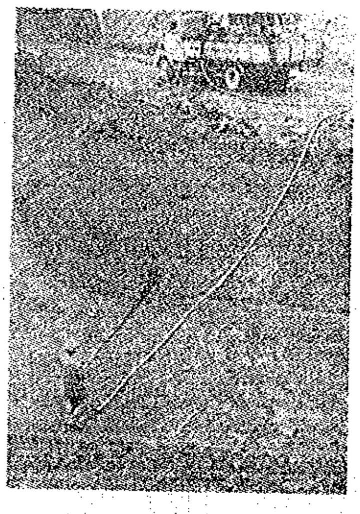


Diagram 3-23 Seeding Work with Pump (Spraying Work)

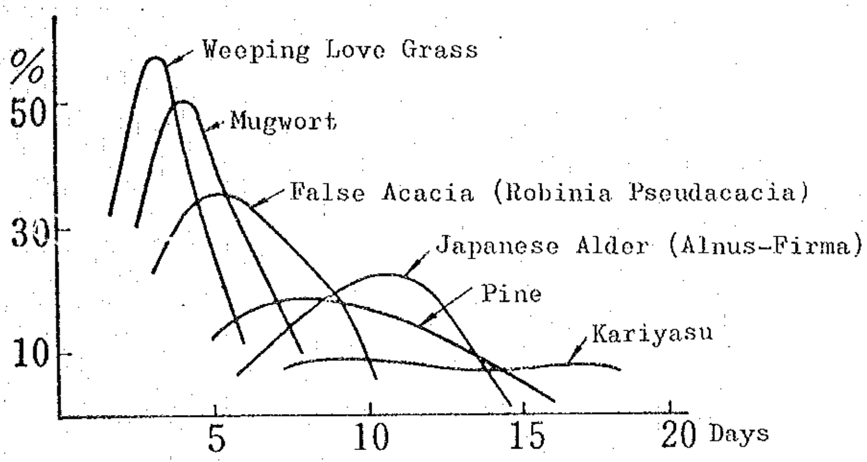


Diagram 3-24 Germination Conditions of Grasses for Hillside Planting Work

Conditions of germination are highly dependent on the environmental conditions of the particular time. Under normal conditions, plants such as Weeping Love Grass, Kentucky 31 Fesk, clover, and Mugwort sprout after 2 - 3 days from seeding, and Creeping Red Fesk, as well as Bermuda Grass sprout within a few days. Conventional perennial herbs which exhibit superb continuosity and applicability devastated lands exhibit delayed and ununiformed germination periods. Many leguminous plants such as Acacia among ligneous trees sprout within a few days and exhibit improved initial growth when germination inprovemen treated.

2) Ease of Breeding

Good quality seeds with strong germinative power must be promptly obtained in large volumes, since abundant vegetation are desired to sprout and grow within a short period of time from seeding during replanting of devastated land. Imported pasture generally exhibit these features among herbs. Additionally, Miscanthus, Mugwort, and Giant Knotweed exhibit good features among perennial herbs, as well as False acacia, Japanese alder (Alnusjaponica and Alnus firma), Oleaster, and pine also exhibit identical features among ligneous plants.

3) Forms of Growth

Forms, periods, and heights of growth are one of the essential conditions to be considered for the selection of the plant to be employed for replanting.

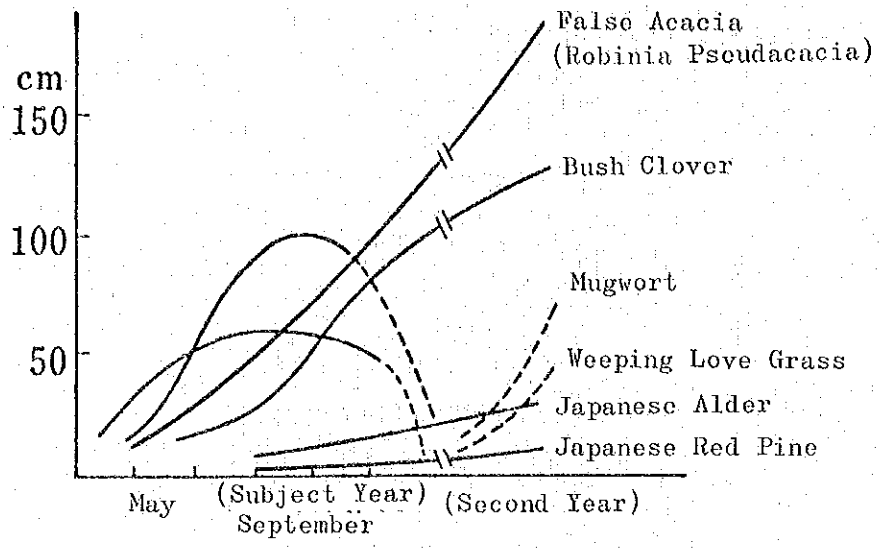


Diagram 3-25 Initial Growth of Plants for Replanting

Growth forms may be segregated into plants which grow in branching at roots fasciculate form such as vegetative plants (e.g. rice and Cyperus microiria), procumbeats or creeping (e.g. sod), erect type (e.g. mugwort), and rhizome plants (E.G. Kentucky 31 Fesk). Procumbeats and rhizome plan exhibit the greatest slope stabilization among the aforementioned. Additionally, there are winter grass type which grow during the colder spring and fall seasons as well as summer grass type which grows well during the hotter summer seasons as characteristics for growth periods. Seed mixtures are blended mainly with the former of the two when planting in the spring or fall or in colder areas, and the latter during the summer or in warmer areas.

There are perennial and therophyte herbs among plants. Therophyte plants not only hamper to the growth of other plants by oppressing with its rapid initial growth, but also exhibit rapid decrease in surface stability when they decline. Therefore, replanting works must be performed employing perennial herbs.

As far as the height of growth is concerned, wild grass such as miscanthus and mugwort grow tall, and oppress other vegetation when they begin to grow densely. Kentucky 31 Fesk and Weeping Love Grass among imported species grow relatively tall and produce vegetative overlay in a short period of time. However, close attention should be paid the seed quantity and mixture proportion of these plants per area unit since they may overgrow and oppress other vegetation. Procumbeats such as Bermuda Grass, Clovers, and sods grow low on the ground and do not oppress or hamper the growth of other vegetation.

4) Other Features

Desirable features of grass and trees for slope seeding work aside from the aforementioned are good growth on infertile and arid land, improvement of soil fertility through root module bacteria function and desease resistance.

Strip Seeding Work

This is a method applicable for relatively gentle gradient places where a mixture of seed and fertile soil are manually sown in ditches cut away on hillsides or steps. However, this method may also be applied on hillsides with fairly steep gradient since sowing seeds are secured in the cut ditches.

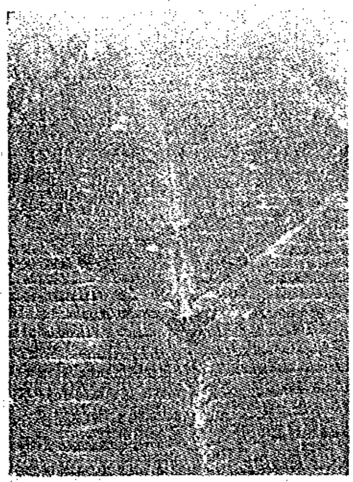


Diagram 3-26 Strip Seeding Work

Procedures most applicable and economical should be selected with the ditch depth and width in consideration, since the amount of soil investment differ depending on the soil condition of the work site.

Slope Seeding Work

This is a method applicable for relatively gentle incline slopes with good soil condition. However, this method may also be applied on steep slope hillsides when protective measures such as covering work

are executed immediately seeding for the prevention of seed washouts as well as for the preservation of moisture for germination.
This work may be performed efficiently by spraying with pumps when the work site can be reached with vehicles. In such case, there are methods where the seeds, fertilizers, and adhesives are sprayed simulatneously, and where they are sprayed individually.
The seeding quantity varies depending on the condition of the land as well as the type of seeding mixture, and therefore, should be determined with the anticipated numbers of germination of 3,000 to 10,000 per square meter and the rate of germination in consideration.

Section 7 Planting Work

Planting works are executed for the early stage restoration of the disaster prevention effects of the forest which are lost on devastated lands with the growth of trees by planting seedlings.
The conditions for the growth of trees may be prepared to a certain extent with the various aforementioned hillside works. However, though, soil nature of such work site normally exhibit far poorer soil nature compared to normal plantation even after the execution of hillside works. Therefore, the soil nature, topography, and climate of the work site as well as the vegetation in the vicinity should be closely studied when making determinations for the planting plan. Improvement measures such as soil amendment and fertilization must be applied in order to be able to expect positive early stage growth and maturity of the vegetation.

1. Planting Plan

Land Classification

Lands are classified in accordance with the land conditions at the work site in order to select the most applicable species and planting quantity of vegetation. Planting plans for the individual regions are as described in the following.
Important trees may be planted when risks for flooding as well as silt wash outs are not evident since such lands exhibit good soil conditions comprised with silt deposited by fixing of channel during the formation of the stream.

Silt deposit part within collapse are formed with silt produced by surface fall-offs from above as well as by soil generated with grading works. Therefore, soil conditions in such land are relatively good, and superb tree growth may be expected.

Denuded zones within devastated lands lack surface soil and have bad conditions such as steep gradient, dryness, and frost heaving. Therefore, hillside planting plans must be designed carefully with these matters in consideration. Additionally, improvement measures such as fertilizing, soil dressing, and mixed planting of soil improving trees must be taken progressively in order to facilitate the development of vegetation.

Tree Species for Planting

Tree species are selected in accordance with the aforementioned land classification, based on the "appropriate tree for the appropriate soil" principle. However, though, it is required that the planted species fulfill the following conditions:

1) Abundant growth and thrift.

2) Broad and deep rooting with large soil securing ability.

3) Sterile soil tolerance

4) Strong resistance against negative factors such as aridity, frost, and insect damage.

5) Those which soil improvement effects may be expected

Generally broad leaved trees such as Japanese alders, leguminous plants, and willows exhibit the aforementioned features. However, though, pine tree which is coniferous is also widely employed. Pine trees have been conventionally used widely as the principal forest crop.

(1) Principla Forest Crop

Japanese Red Pine, Japanese Black Pine, Pinus Rigida, Yezo Spruce, White Fir, Japanese Cypress, Zelkova Serrata

(2) Soil Improving Rree

Japanese alder, Mountain Aldes, HIMEYASHABUSHI, (Alnuspendula) Wax-Myrtle (Myrica rubra), willow, Acacia, ITACHIHAGI (Amorpha frution a kind of Lespedeza bicolor), Oleaster, Deutzia (Deutzia crenata)

Furthermore, trees differing in characteristics should be mix-planted in order to reduce various hazards for the development of a highly resistant forestes described in the following.

1) Shallow and deep rooted trees.

2) Principal forest crops and soil improving trees.

3) Coniferous and broad leaved trees.

4) High and low growing trees.

Planting Quantity

Planting quantity varies depending on the species, planting method, gradient of the planting site, and the soil conditions, but should be limited to the standard of 3,000 - 5,000 trees per ha. in good soil condition areas such as soil deposit part within hillside landslide as well as in stream bed, and to 8,000 - 10,000 trees per ha. in poor soil condition areas such as denuded part within hillside landslide. Planting quantity of soil improving trees does not vary with the principal forest crop, but is required to pay close attention to the layout and number of mix-planting, since soil improving trees exhibit rapid initial growth, and therefore, may be oppres the principal forest crop.

2. Planting and Care

(1) Planting

Healthy seedlings with good surface and root system balance should be selected for erosion control planting dur to the poor soil conditions of work site. Additionally, close attention should be paid to prevent from damaging the seedlings during digging out, temporary planting, and transportation.

Seedlings should be planted somewhat deeply in a largely dug out planting hole with the application of soil dressing and fertilizer as deemed appropriate. On the contrary, seedlings must be planted high in low damp ground or in high ground water areas to prevent rainwater accumulation in the planting holes.

(2) Care

Planted trees must be protected with straw litter around the bottom of the trunk in order to prevent desiccation along with weeding until they survive and commence to grow. Additionally, close attention must be paid to the opperssion against the principal forest crops when mix-planted with soil improving trees. This is particularly important when light demanding trees such as pines are used for the principal forest crop. In such case, the number of salvage cutting frequencies needs to be increased for frequent removal. Furthermore, additional fertilization should be performed to facilitate growth when found to be poor. Replanting should be performed rapidly when bare ground surface caused by dead standing trees are found since devastation may expand again from such ground. Moreover, partial foundation work damages must be immediately restored since such conditions may result in excessive disasters.

CHAPTER IV TORRENT WORK

Section 1 Basic Hydraulics for Torrent Work Disigning

1. Hydrostatic Pressure

(1) Hydrostatic Pressure

Rest water which is not flowing is called static water. The pressure of the weight of the water under the static condition is called the hydrostatic or hydraulic pressure. This force is uniform on the surface, and the total resultant of force effecting the entire plane is called the overall hydraulic pressure.

1) Intensity of Hydrostatic Pressure

Hydrostatic pressure is expressed per unit areas since the force is distributed on the plane. The following equation is derived when the hydraulic pressure is uniform at any points on plane A, with the overall hydraulic pressure is P and the force of the water is p:

$$p = \frac{P}{A} \qquad (4\text{-}1)$$

Total hydraulic pressure is expressed in kg, t and other hydraulic pressures are expressed with unit kg/cm^2 and t/m^2, etc. The overall hydraulic pressure effecting a certain plane may be considered as a concentrated force equivalent to the hydraulic pressure effecting the plane. Therefore, it is necessary to define the effecting point of the overall hydraulic pressure as well as its strength.

2) Hydrostatic Pressure Characteristics

Hydrostatic pressures effect the plane vertically. Furthermore, the hydrostatic pressure p is proportional to the depth of the water h. Therefore, with the unit weight① of the water as w; it may be expressed by the following equation.

$$p = wh \qquad (4\text{-}2)$$

Additionally, the hydraulic pressure on a certain point in the water acts constant force in all directions. (Diagram 4-1)

*① Weight per cubic unit.

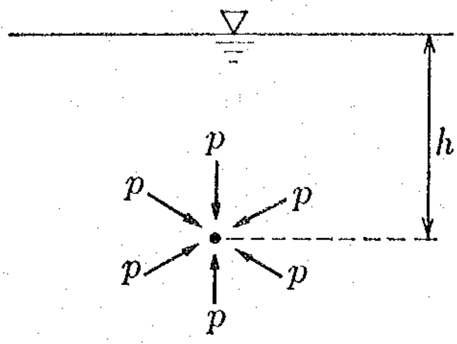


Diagram 4-1 Hydraulic Pressure On A Certain Point in the water

(2) Hydraulic Pressure Effecting A Level Plane

1) Hydraulic Pressure On A Horizontal Plane

The intensity hydraulic pressure acting on horizontal plane such as the bottom of a container is uniform at any point on this plane. Therefore, the overall hydraulic pressure P may be derive from the following equation where the depth of the water is h, whereas $p = wh$, and the area as A:

Equation $$P = pA = whA \qquad (4\text{-}3)$$

The point where the overall hydraulic pressure effects this surface is at the center of figure①. (Diagram 4-2).

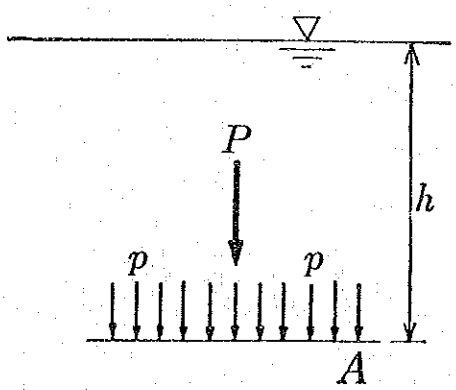


Diagram 4-2 Hydraulic Pressure on A Horizontal Plane

① Center of gravity on a plane figure.

2) Hydraulic Pressure on A Vertical Plane

Hydraulic pressure effects a plane perpendicularly and is proportional to the water depth. Therefore, the distribution of the hydraulic pressure effecting the rectangular zone of the perpendicular height h becomes ΔABC as illustrated in daagram 4-3. Therefore, the overall hydraulic pressure P is described by the following equation with the width of this zone as b.

equation $$P = \frac{1}{2} wbh^2 \quad (4\text{-}4)$$

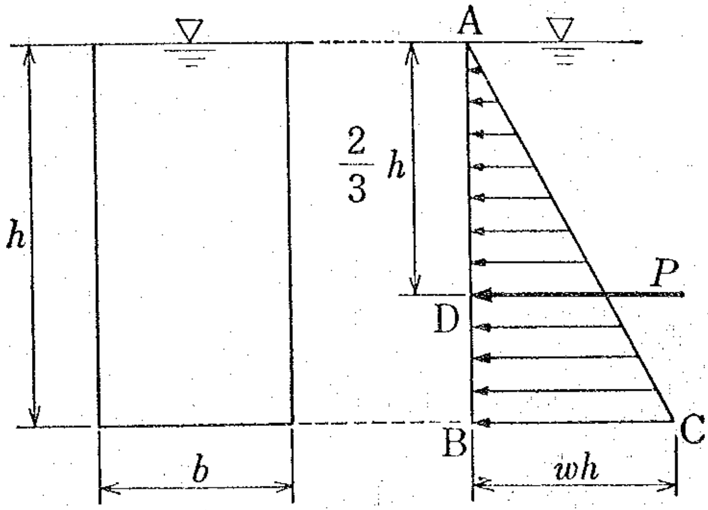


Diagram 4-3 Hydraulic Pressure on A Vertical Plane

The water depth at the effecting point D is 2/3 h since the line of application line of the overall hydraulic pressure P works its way vertically towards the plane AB through the center of figure of the ΔABC.

3) Hydraulic Pressure on A Sloped Plane

The distribution of the hydraulic pressure strength effecting a sloped plane AB with gradient incline 1: n as illustrated in diagram 4-4 becomes ΔABC with ∠B as a right angle. Since AB = $h\sqrt{1 + n^2}$, = wh at this point, the overall hydraulic pressure P may be derived from the following equation with the width of the area as b.

equation $$P = \frac{1}{2} wbh^2\sqrt{1 + n^2} \quad (4\text{-}5)$$

The water depth at the (effecting point D is 2/3 h since the line of the application of the the overall hydraulic pressure P works its way vertically towards the sloped plane AB through the center of figure of the ΔABC.

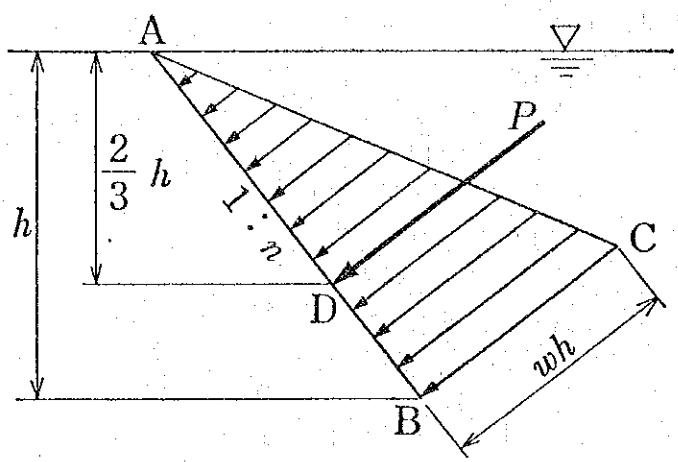


Diagram 4-4 Hydraulic Pressure on A sloped Plane

2. Stream Velocty, and Movement of the Sand and Stone

(1) Basic Characteristics of the flowing water

1) Flow area, wetted Perimeter, and Hydraulic Radius

The movement of water is called flow. The cross section of the channel perpendicular to the direction of the flow is called the transverse channel section, and the area or the portion of the transverse channel section with water is called the flow area (or cross-sectional area of flow). Additionally, the length of the transverse channel section contacting the water is called the wetted perimeter.

Furthermore, the result derived by dividing the flow area A by the wetted permeter P is called the hydraulic radius R. The following equation is the formula for the aforementioned:

equation $R = \frac{A}{P}$

In Diagram 4-5(b) $A = h(b + nh)$ (4-6)

$P = b + 2h\sqrt{1 + n^2}$

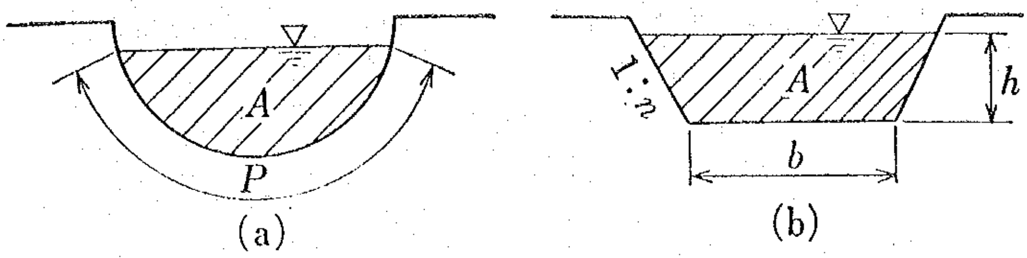


Diagram 4-5 Flow area, Wetted Perimeter and Hydraulic Radius

Flow area is expressed in units of m^2 or cm^2, whereas the wetted perimeter and hydraulic radius are expressed in units of m or cm.

2) Velocity and Discharge

The water in an open channel ① flows from a higher elevation to a lower elevation. The speed of this flow is called the flow velocity. This flow velocity is expressed as the flow distance per a unit time. The flow velocity v flowing the distance L within the time t may be derived from the following equation, and is expressed in units such as m/s and cm/s.

equation $$v = \frac{l}{t} \qquad (4\text{-}7)$$

Generally, this flow velocity differs at every point of the flow area. Therefore, the average flow velocity for the overall flow area is considered, and is used as the average flow velocity of that particular flow area. Additionally, the volume of water flowing through the flow area within that time is called the discharge. The following equation may be derived with the average flow velocity as v, the flow area as A, and the discharge as Q.

equation $$Q = vA \qquad (4\text{-}8)$$

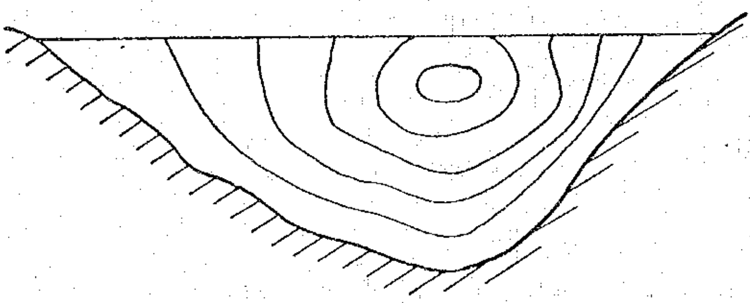
The discharge units are expressed in m^3/s and l/s, but also may be expressed in t/s since a volume weight of water is $1t/m^3$.

① Open channels are water ways such as streams rivers, and artificial canals possessing free water surface with the atmosphere.

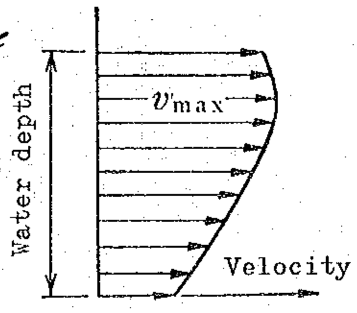
(2) Velocity Measurement

1) Velocity Distribution in an Open channel

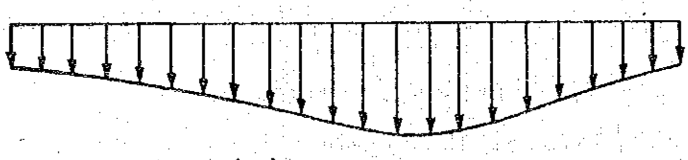
The velocity at various points of transverse channel sections is not constant. This inconsistency is due to effects of the roughness which express is the roughness of the waterway walls, the shape of the transverse channel section, channel sinuosity, and the depth of the water. Diagram 4-6(a) illustrates the distribution of the flow velocity at the transverse channel section as a flow velocity contour curve. The flow velocity generally become greater further away from the channel sides and bottom. This phenomenon is the resultant of the reduced effects of friction with the aforementioned sides and bottom. Therefore, the maximum velocity Vmax is generated at a point slightly below the surface in the vicinity of line of maximum depth. Additionally, the surface flow velocity is smallest along river banks as illustrated in diagram 4-6(b), and increases further distance away from river banks until it becomes greatest in the vicinity of line of maximum depth. Furthermore, the curve which describes the flow velocity distribution on a perpendicular line through a certain point of the transverse channel section is called the vertical flow velocity curve. The vertical curve in the vicinity of line of maximum depth generally becomes similar to that illustrated in diagram 4-6(c). Hereby, the maximum flow velocity vmax is generated 0.1h - 0.4h beneath the surface of the water, and the average flow velocity v is generated at a depth of 0.5 - 0.65h from the surface.



(a) Velocity Contour Curve



(c) Vertical Velocity Curve



(b) Surface Velocity Curve

Diagram 4-6 Velocity Distribution

2) Velocity Measurement with a Current Meter

A current meter is a device used to measure the flow velocity. The measurement is accomplished by lowering a propeller into the desired depth of the water current in order to count the number of revolutions within a specified time. The flow velocity may be derived from the following equation since this equation is generally applicable to express the relation between the number of revolutions and the velocity of flow.

equation $$v = aN + b \qquad (4\text{-}9)$$

Note: v: velocity (m/s), a,b: constants,

N: number of propeller revolutions per second.

*a and b values are the exclusive constants of the current meter and should be primarily derived through experimentation.

Current meters are available with cap form propellers with rotating shafts perpendicular to the flow, and thos illustrated in diagram 4-7 have with shafts parallel to the flow.

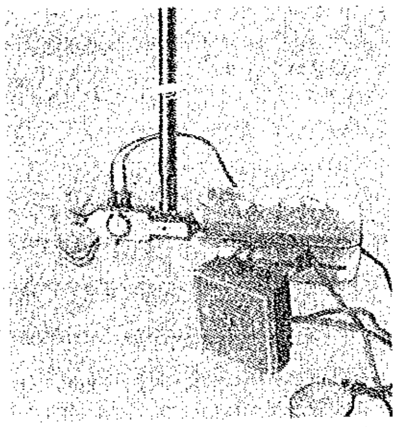


Diagram 4-7 Current Meter

Transverse channel sections are segregated with perpendicular lines for transverse survey in fairly large waterways with large discharge as illustrated in Diagram 4-8, in order to derive the average flow velocity in the individual segregated sections through in equation with the velocity measurements taken at several points on the perpendicular line running through the center of these sectors.

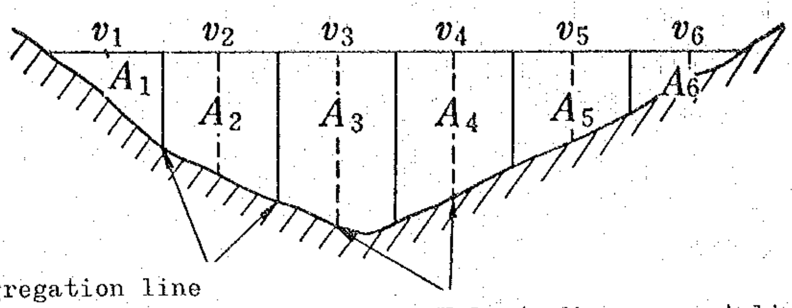


Diagram 4-8 Measurement of Water Channel Section

The flooweing equations are available for deriving the average velocity of the segregated sectors when the velocity in depths 0.2h, 0.6h and 0.8h from surface are v0.2, v0.6 and v0.8, respectively, with the depths of the perpendicular line running through the center of the segregated sectors at h. The following are the applicable equations:

1) 1 Point Method (equation) $v = v_{0.6}$

2) 2 Point Method (equation) $v = \frac{1}{2}(v_{0.2} + v_{0.8})$ (4-10)

3) 3 Point Method (equation) $v = \frac{1}{4}(v_{0.2} + 2v_{0.6} + v_{0.8})$

Generally, the 2 Point Method which is relatively simple with high accuracy is used. When particularly accurate values are required, vertical velocity curves are drawn for the individual section to derive the area surrounded by these curves and the vertical axis and to divide this by the depth to derive the average velocity of each section.

The average velocity of the overall channel section is derived by dividing the discharge obtained with the "Velocity method" explained in the following "Discharge Measurement" by the overall flow area.

3) Velocity Measurement with Floats

This is a method where the velocity is derived by equation $v = \frac{l}{t}$ based on the time measured for a float to float downstream for a certain distance. Among the various floats available, there is the surface float which floats on the water surface and the pole float provided with a weight to maintain an upright position in order to measure the average velocity of the surface and underwater currents.

(3) Mean Velocity Formula

Although various formulas are available to derive the mean velocity, it is best represented by the Chézy formula described below:

$v = C\sqrt{RI}$ (4-11)

Where; v: mean velocity within the flow area (m/s), C: Velocity coefficient, R: hydraulic radius(m), I: water surface slope.

Various empirical formulas are available for deriving the velocity coefficient C:

1) Bazin's Old Formula

equation

$$C = \sqrt{\frac{1}{\alpha + \frac{\beta}{R}}} \qquad (4\text{-}12)$$

Where; C: velocity coefficient, R: hydraulic radius (m), a, β: coefficient of roughness

Generally, $\alpha = 0.0004$ and $\beta = 0.0007$ are applied for torren streams. Details of coefficient of roughness are described in Chart 4-1.

Chart 4-1 Coefficients of roughness for the Bazin's Old Formula

Classi-fication	Channel	α	β
I	Cement coated or plane finished wood channels.	0.00015	0.0000045
II	Smooth surfaced cutting stone, brick, or unfinished wood channels.	0.00019	0.0000133
III	Rough stone or stone pitching channels.	0.00024	
IV	Natural earth and stone channels.	0.00028	0.000350
V	Channels washing down rubbles and rough gravel.	0.00040	0.000700

2) Bazin's New Formula

equation

$$C = \frac{87}{1 + \frac{r}{\sqrt{R}}} \qquad (4\text{-}13)$$

Where; r: coefficient of roughness

Generally, r = 1.75 is used for torrent streams. Details of coefficient of roughness are described in Chart 4-2.

Chart 4-2 Coefficient of roughness for the Bazin's New Formula

Classification of Channel	Smooth cement coated or plane finished wood channels.	Concrete, brick, or rough wood channels.	Rough stone masonry or rough brickwork channels.	Uniformed soil channels.	Normal soil channels.	Rough soil channels.
γ	0.06	0.16	0.46	0.85	1.30	1.75

(4) The critical Velocity and the equilibrium Slope of Gravel

The critical Velocity of Gravel

The current tends to impact and wash off the gravel on the streambed. In turn, the gravel tends to resist this force by the friction with the streambed. The larger the friction at this point, the longer gravel remains, but gravel is washed off when this friction is small. The magnitude of this impact force to washout the gravel is proportional to the square ($*^2$) of the velocity. Therefore, the movement of the gravel is wholly contingent on the velocity. This is why the maximum velocity for the gravel to remain on the streambed is called the critical Velocity of the gravel. The following is the simplified formula to derive the aforementioned: equation

$$v_g = K\sqrt{b} \qquad (4\text{-}14)$$

where; vg: Critical Velocity of the Gravel (m/s),

k: Constant 3.7 - 5.3/average 4.5

b. The length of gravel in the direction of the flow(m).

Assuming that the streambed is comprised of layers of packed gravel, the flowing water tends to wash these gravel layers with a somewhat leveling force. This force is called the tractive force and becomes larger with an increased hydraulic raulic radius and water surface slope. The velocity becomes larger in accordance with equation (4-11) when the hydraulic radius and the water surface slope are large. Therefore, the tractive force too, is closely related to the magnitude of the velocity.

2) Equilibrium Slope

The velocity of flow decreases when gravel is mixed in the flowing water.

Therefore, when the containing gravels of various sizes reaches a certain point, gravel on the streambed with critical velocity smaller than the speed of the flowing water is washed away. Since the velocity becomes smaller at this point, washed down gravel with a critical velocity larger than the speed of the flowing water settle down on the streambed. As previously described, the flowing water not only washes away gravel, but settles gravel also. The strembed becomes eroded when the volume of the washed out gravel is larger than the settlement, and sand depositting occurs when the washout is smaller. Furthermore, the streambed slope remains the same although the gravel may be changed when the aforementioned both volumes are equal.

This unchanging slope is called the equilibrium slope. In order to derive this, it becomes;

$$C\sqrt{RI} = vg = K\sqrt{b}$$

when considering equations (4-11) and (4-14) to be equal.

With both sides squared, it becomes;

equation $$C^2RI = v_g^{\ 2} = K^2b \qquad (4\text{-}15)$$

Therefore, the slope l where the gravel with the diameter b becomes stabilized is described by the following equation:

equation $$I = \frac{K^2b}{C^2R} = K'\frac{b}{R} \qquad (4\text{-}16)$$

Where; $K' = \frac{K^2}{C^2}$

Generally, the discharge is less and the size of the gravel on the streambed is larger in the upstream compared to the downstream. Therefore, the equilbrium slope is steep in the upstream and relaxed in the downstream as described in Equation (4-16). But when observed on separate points, the equilibrium slope varies due to the states of gravel intermixture and discharge variation.

3. Discharge Measurement

When executing or designing stream work, it is necessary to know the discharge of the stream both for normal and particularly for flood conditions. The following are the often practiced methods among the various discharge measurement or approximation methods:

(1) Velocity Method

This is a method where the velocity is derived with the product of the measurement taken of the mean velocity and the flow area.
The desirable measurement point is where the channel is straight and the channel width and depth are constant. The velocity is measured with the most compatible method for the condition of the stream and the purpose of the measurement. The flow area is to be derived by cross sectional survey of the channel.
For streams with broad streambeds and large, the cross section of the stream is segregated into several section as illustrated in Diagram 4-8 in order to derive the cross sectional area and the mean velocity individually. Then, the discharge for the individual sections are derived with the product of the cross sectional area and the mean velocity.
These are then summed to obtain the total discharge of the stream.
The following equation describes the above as a formula:

equation $$Q = A_1v_1 + A_2v_2 + A_3v_3 + \ldots\ldots + A_nv_n \qquad (4\text{-}17)$$

Where; Q: total discharge, (m^3/s)

A_1, A_2, A_3.........., An: Individual cross Sectional Area (m^2),
Vl, v2, v3 ...., Vn: Individual Sectional mean velocity (m/s)

(2) Method with the Spillway of A Check Dam

When the water storage is held in the upstream of the dam without sand depositting, the discharge is derived by the measurement of the depth of water overflowing from the spillway. When the cross section of the spillway is a trapezoid with 10% side grading, the discharge may be derived by the following eaution:

equation $$Q = (1.77B + 1.42h)h^{\frac{3}{2}} \qquad (4\text{-}18)$$

where: Q: discharge (m^3/s), B: length of the crown of the spillway (m), h: overflow water depth (m)

When sand is deposited up to the crown of the dam in the upstream, the discharge is derived from the product of the flow area and mean velocity measurements taken on the overflow from the spillway.

(3) Flood Level Marking Method

This is a method where the maximum discharge during floods are estimated from the flood markings. Although the method is simple to use, the result also tends to be inaccurate. The flood level is estimated by surveying the channel immediately after the flood as well as by checking the water coverage states of the vegetation in the vicinity of stream banks, surface soil washout, contamination by muddy water, and caught on trash locations, etc. Following these, the channel is cross sectionally surveyed to obtain the flow area. Additionally, the water surface slope or the streambed slope is derived from the flood stage in order to derive the mean velocity by the mean velocity formula. Then, the maximum discharge during the flood is derived by the product of the flow area and the mean velocity.

(4) Method by Rational Formula

This is a method used for determining the cross section of the dam spillway and estimating the maximum flood discharge, and discharge is derived

equation: $$Q = 0.2778frA \qquad (4\text{-}19)$$

Where: Q: maximum flood discharge (m^3/s), f: coefficient of runoff
r: maximum rainfall per hour (mm/h), A: catchment area (km^2)

(5) Method by Specific discharge

The value of the discharge at a certain point on the stream or river divided by the catchment area is called the specific discharge. Therefore, the flood discharge may be approximated when the specific discharge during the flood and the catchment area are known.

The formula for the this is described below:

equation $$Q = Aq \qquad (4\text{-}20)$$

Where; Q: Flood discharge (m^3/s), q: Specific discharge during the flood, the flood ($m^3/s/km^2$), A: Catchment area (km^2).

Although the Specific discharge should be derived for the individual points, the specific discharge for torrent streams during floods are generally as listed in Chart 4-3.

Chart 4-3 Specific discharge of torrent streams During Floods

Catchment Area (km^2)	0 ~ 10	10 ~ 20	20 ~ 40	40 ~ 60	60 ~ 80	80 ~ 100
Specific discharge ($m^3/s/km^2$)	25	20	15	12	10	8

① Coefficient of run-off: the depth of run-off is the total volume of water run-off from the watershed during a certain period divided by the area of the watershed. This figure divided by the precipitation is called coefficient of run-off.

Section 2 Objectives and Types of Torrent Work

1. Objectives of Torrent Work

Large volumes of gravel, soil and sand are transported downstrem during floods of torrent streams, and therefore, sedimantation zone tends to cause disasters. Therefore, torrent work has the following objectives.

1) To use as foundation of hillside works and to enhance natural restoration of the hillside landslide area, by securing the foot of hillside landslide area.

2) To prevent the erosion of streambeds and stream banks as well as to prevent hillside landslide.

3) To form sound and stable stream by preventing the washout of unstable sediment as well as stabilizing the stream by depositting the run-off sediment from the upstream devastated land and by controlling the run-off sidement to downstream.

2. Types of Torrent Work

The velocity must be reduced to decrease the erodibility of the flowing water in the torrent. The streambed gradient and the water depth must

be reduced. This is why check dams are erected. They reduce the slope of the stream bed by depositting in their backwater area.
Additionally, the streambed width increases and the water depth reduces along with the sedimentation (diagrams 4-9 and 10 refer).
Among structures erected mainly for prevention of such longitudinal erosions are sediment control dams, ground sill work ①, and small check dams ②. Since these structures are erected across the stream, they are generally called cross works.
In the other hand, when the turbulent flow in the stream becomes excessive, the water current driftes and erodes the bank, risking the danger of hillside landslide. Among the structures erected for prevention of this lateral erosion are revetment works ③ and spur dyke ④. These structures are erected on the torrent banks along the stream, and therefore are called longitudinal works.
Both cross and longitudinal work should be done in conjunction with each other in order to stabilize the torrent stream since lateral and longitudinal erosion generally take place simultaneously in torrents Stone pitching water channel work and concrete water channel work are done at times to increase the resistance of the streambed against the erosion by the water flow. Furthermore, in the downstream of the torrent, channel work ⑤ is also done at times to protect the stream bank and to secure the streambed.

Section 3 Check Dam (Sediment Control Dam)

1. Objectives and Types of Check Dams

(1) Purpose

Check dams are structures built across the stream with the following objectives.

1) To reduce the streambed gradient to prevent longitudinal erosion. This is accomplished by erecting a tall independent dam or with low multiple stepped dams as illustrated in Diagram 4-9.

*① Explained in details later
② Small dam-like structures erected in the upstream regions of the torrent.
③, ④, and ⑤ are explained in detail later.

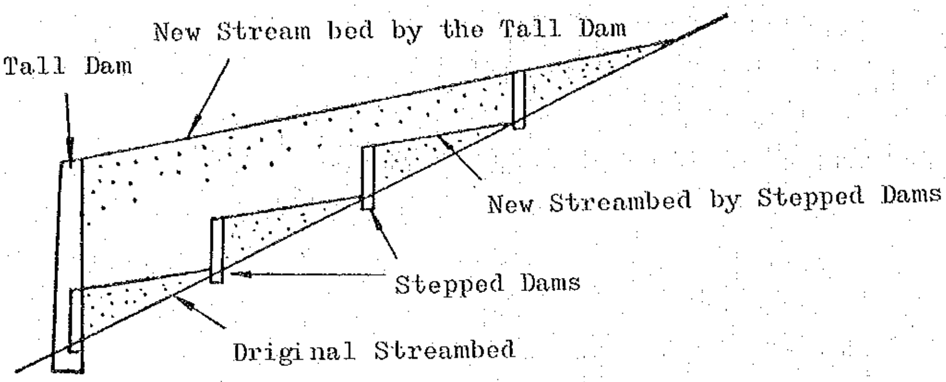


Diagram 4-9 Reduction of the Streambed Gradient

2) To prevent the hillside from collapse by securing the foot of hillside. As illustrated in Diagram 4-10, the streambed is elevated to secure the unstable foot of hillside in order to prevent the hillside from collapse as well as to preclude expanding of collapse from expanding.

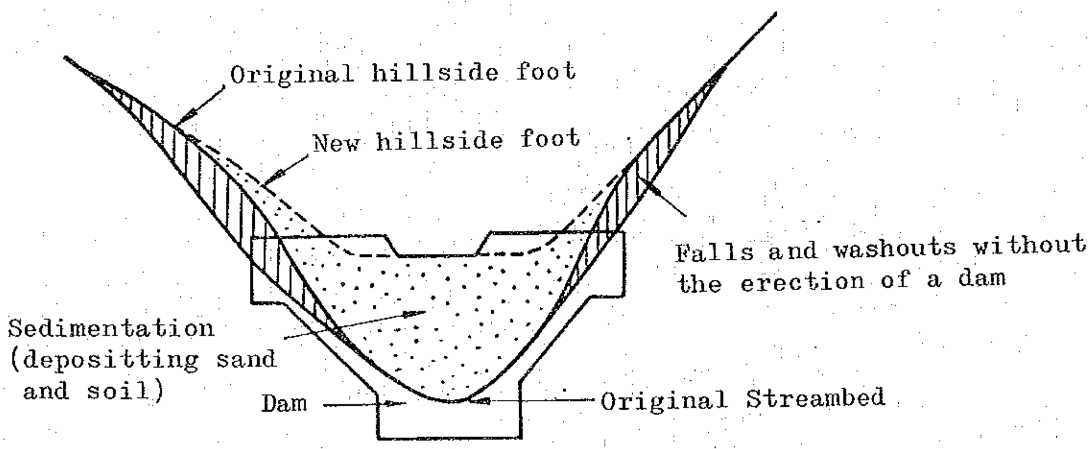


Diagram 4-10 Securing of hillside foot

3) To check the sand and gravel washed down from the upstream and controls the funoff to the downstream. Dams for such porpose are called check dams and can restrain as well as reduce the force of mud flows. The height of dams are generally constructed high as illustrated in Diagram 4-11.

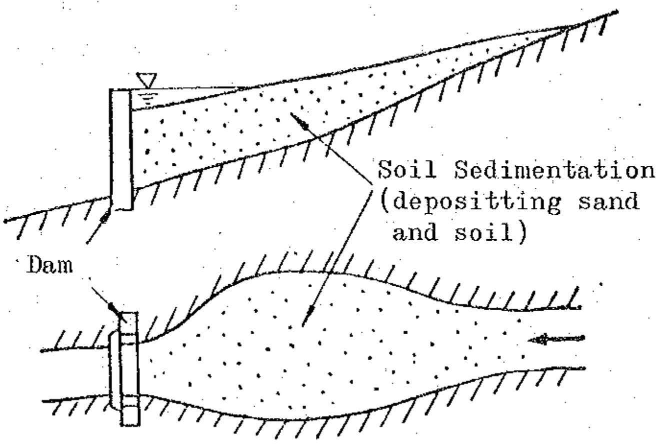


Diagram 4-11 Check of soil and sand

4) Turbulent flows are prevented and streambeds are secured in sedimentation zone. Low stepped dams are erected as illustrated in Diagram 4-12 in order to preven lateral erosion by regulating the channel.

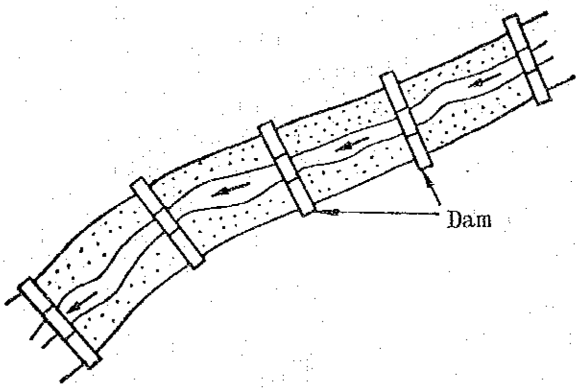


Diagram 4-12 Securing the Streambed

(2) Types

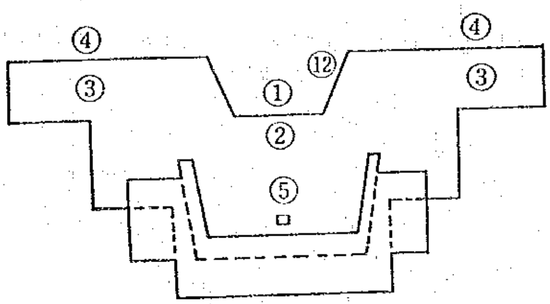
The types of check dams are generally classified in accordance with their structural material, form, and type of resistance against external force.

1) Classification in accordance with the structural material:
Wet masonry, Dry masonry, Mixed masonry, concrete, boulder concrete, reinforced concrete, concrete frame, steel, cylinder, wood and earth, etc.

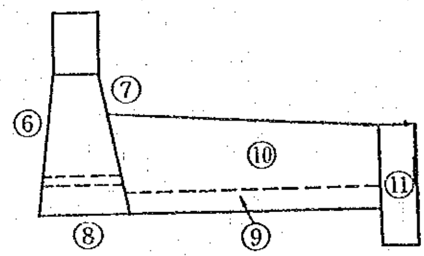
2) Classification by the form:
Straight line, arch, and buttress, etc.

3) Classification by the resistance type against external force:
Gravity and arch system, etc.

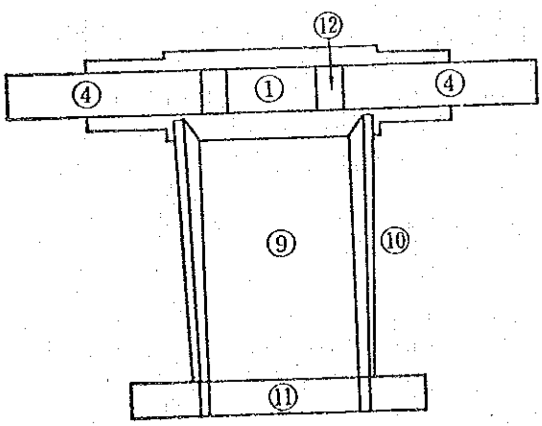
(3) Individual check Dam Section Nomenclatures



(a) Front View



(b) Side View



(c) Plane figure

① Spillway (Overflow section)
② Spillway Crown (Crown)
③ Wing
④ Wing Crown
⑤ Drainage hole
⑥ Upstream slope
⑦ Downstream slope
⑧ Base of Dam
⑨ Front Apron
⑩ Side Wall
⑪ Vertical Wall
⑫ Side of Spill Way

Diagram 4-13 Individual Check Dam Section Nomenclatures

(4) Major Check Dams & Their Features

1) Concrete Dam

This is a dam constructed by forming the shape of the dam with wood or steel forms and placing concrete within the forms. The dam body may be made into an uniformed structure with this method. Additionally, dams made with this method exhibit superb durability, and therefore, tall dams can be made. This is why this method is most widely used lately. Gravity type concrete dams are most representative of this method. This dam resists external force with its own weight and form is generally straight line and may be constructed on gravel layers or soft rockbeds upon treatment to secure the base ground.

Arch type concrete dams relate the external force to the rockbeds in the both sides with the arch effect to resist external force. The dam body may be constructed thinner than the above gravity dams with this method, but the design and construction become more complicated. Therefore, arch dams are constructed when the foundation ground as well as the both sides rockbeds are rigid and the valley is narrow compared to the height of the dam.

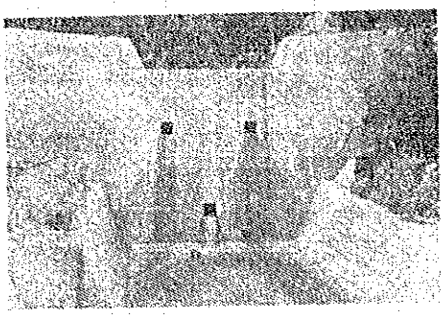


Diagram 4-14 Concrete Dam

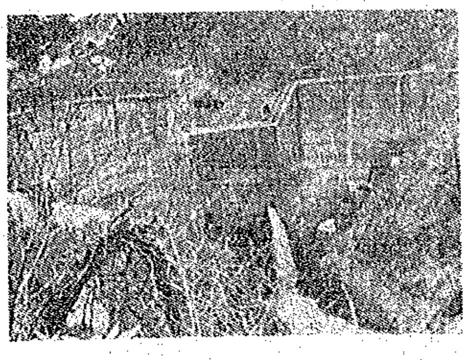


Diagram 4-15 Arch Type Concrete Dam

2) Steel Dams

The base and wings of this type of dam are constructed with concrete. In top of this base, buttress frames comprised of H framed steel assembled into inverted V shapes are placed every 2 meters. The dam body is then formed by placing L frame steel in screen form onto the front of these frames.

The material quality is uniformed since the structural members are factory produced. Additionally, this construction work requires no exclusive technology since the work is accomplished by simply assembling the materials. Therefore, the work may be performed efficiently in a short time period and may be used in undeveloped areas using existing transportation systems. Furthermore, harmless sand is sifted and run-off with the screen structure and only large gravel and boulders washed down by floods are intercepted. Therefore, the hydraulic pressure may be reduced with this large permeability. Due to these reasons, many construction examples are seen despite the short period since its development. However, this method is not suitable for high acidity water due to the corrosiveness of the steel members and securing jointbolts. Moreover, this dam is fragile against impacts as well as force from the diagonal direction. Therefore, this method cannot be done in areas prone to large scale mud flows as well as in curved flow areas.

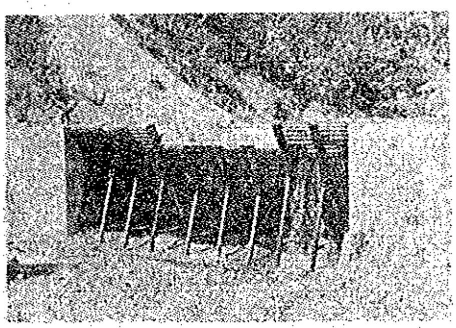


Diagram 4-16 Steel Dam

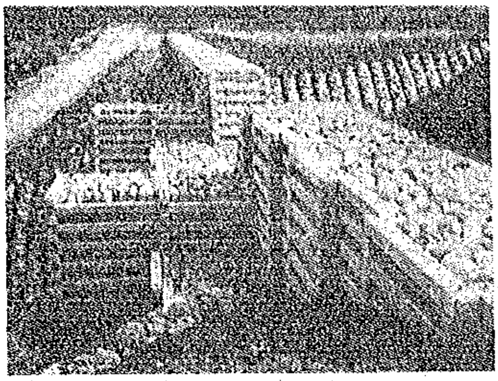


Diagram 4-17 Concrete crib dam

(3) Other Dams

Concrete crib dams are constructed on concrete foundations with square, concrete columnar blocks assembled into a well crib shaped frame filled with boulders. This has improved durability over the wood crib dam and exhibits features identical to the steel dam for construction. However, blocks in the vicinity of crown tend to become damaged in this method. This method is generally done in minor streams with minimal discharge and abundant gravel or in minor streams within the land slide area.

Check dams employing wire cylinders have been long used in bare mountain areas. This work is accomplished by piling on one or more wire cylinders and stopping them with piles. Since this method exhibits superb flexibility as well as compativility with the scour of the foundation and land subsidence, it is done in minor streams on unstable earth.

However, it should only be used for temporary structure due to corrosion of the steel wire resulting in low durability.

Dry masonry dams are constructed simply by piling stone materials and were previously employed for check dams in minor streams in undeveloped areas. Wet masonry dams are constructed of masonry coated with mortar as an adhesive and internally filled with boulders. Although this type of dam was often constructed since the tall dam could be erected by this type in areas where satisfactory stone materials were available, they have been rarely constructed in recent years.

2. Location and Direction of Check Dams

(1) Erection Location

The following conditions must be satisfied for the erection of a dam:

1) The dam must be erected in a spot with rigid bedrock in the streambed or stream banks. This condition must be satisfied since the dam may collapse if the discharge over-flowing the dam washes out the toe of slope or erodes both sides when the foundation ground is soft.

2) The dam must be erected in a location where the valley width are narrow and the streambed slope upstream is gentle as well as where the streambed width is broad. The length of the dam may be shortened and the costs are reduced when the valley width is narrow. Additionally, large amounts of soil, sand, gravel, and rocks may be depositted when the streambed slope upstream is gentle and when the streambed width is broad.

3) The dam must be erected in the downstream of a confluence when erecting a dam in the vicinity of confluence of stream. Dams should be erected in torrent stream when one of the streams is devastated and should be located in at a point where the confluent current becomes stabilized when both streams are devastated.

4) The point where the estimated sand depositting line intercepts the present existing streambed is the site for the upstream dam when planning dams in steps as illustrated in Diagram 4-9.
Based on these principles, the dam location is determined in accordance with the objective of the dam.
Furthermore, when the objective is to prevent streambed and bothsides banks erosion as well as hillside collapse and expansions, the dam is to be erected downstream in the vicinity of these possible devastated areas. Additionally, the dams are to be stepped when this possible devastated section is long. Moreover, the dam is to be planned downstream of the sedimentation zone when the objective is to deposit run-off sediment from upstream.

(2) Direction of the Dam

The water overflowing the dam generally runs-off at a right angle

direction to the dam. Therefore, when the stream is straight, the dam is to be constructed at a right angle direction to the stream.
Dams should not be located in curved flow section, however, when other sites are not available, it is to be designed at a right angle direction to the tangent of line of maximum depth during floods at the center of the overflow section. Additionally, when the present existing streambed width is narrow and the stream is bent as illustrated in Diagram 4-18(a), the new streambed after the soil sedimentation in the dam often tends to become straight line. Therefore, the direction of the dam must be determined with considerations for the estimated new line of maximum depth.
Furthermore, when planning stepped dams in curved flow areas, the dams are to be designed at a right angle direction to the line connecting the center of the overflow section of the upstream and downstream dams as illustrated in Diagram 4-18 (b).

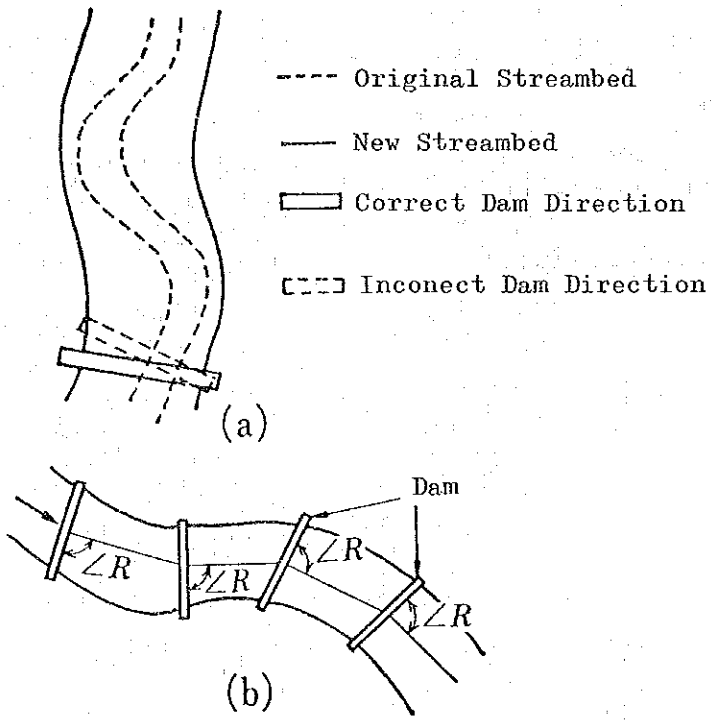


Diagram 4-18 Direction of the Dam

3. Design Accumulating Gradient and Dam Height

(1) Design Accumulating Gradient

One of the objective of check dams is to deposit soil and sand upstream area in order to make the streambed gradient gentle to produce stable stream free from a longitudinal and lateral erosion. It is important to estimate the new streambed gradient. This is called the design accumulating gradient, and the height and location of the dam is determined with this gradient figure.

The accumulating gradient of dams generally are steep when the gravel discharge from the upstream is excessive or when the gravel sizes are large, and are gentle when the discharge is large. Accumulating occurs on steep gradient during large floods, but this gradient gradually becomes relaxed with succeeding intermediate and minor floods. The gradient of the streambed is therefore changing constantly with the repetition of the this phenomena.

The design accumulating gradient for designing dams is determined with examples in the vicinity of the site as references based on the standard 1/2 - 1/3 of the present existing streambed gradient with the streambed comprising gravel sizes and the discharge, etc. taken in consideration.

(2) Dam Height

The height from the base of the dam to the crown of the overflow section is called the dam height, and the height from the streambed prior to construction to the crown of the overflow section is called the effective height. The height of the dam is determined upon considering the objective of the dam construction, ground condition, and the design accumulating gradient, etc. Therefore, the objective must be clearly distinguished as well as the foundation ground of the projected dam construction site must also be thoroughly surveyed when determining the height of the dam. The dam height in accordance with these objectives is described below.

1) Whether to construct a relatively tall dam or several low stepped dams is determined in accordance with the conditions of the project site when the objective is to prevent the erosion of hill side foot in large scale hillside land slide areas. In this case, it is

often more effective to construct several low dams in place of a single tall dam. This is because lower dams are smaller in volume and more economical as well as receive less scour on the downstream toe of slope. However, taller dams are superior to the lower versions as far as the soil sedimentation volume is concerned.

2) Low dams are constructed in steps generally when the objective is to prevent the erosion of the hill side foot of the streambank collapse area.

3) The dam height must be sufficient for the protection of the foundation when the objective is to protect the foundation of structures from scour.

4) The dam height must be as high as possible for the foundation ground and the topographical conditions when the objective is sedimentation.

(3) Sand Sedimentation Volume Calculation

Cross sectional and longitudinal sectional methods are available for derivation of the sand sedimentation volume. The longitudinal section method is explained below.

Sand depositting solid behind the dam AB as illustrated in Diagram 4-19, it is assumed that the longitudinal section becomes ABC. This longitudinal section area is multiplied by the average width of the sedimentation to derive the sedimentation volume.

Where V : sand sedimentation volume (m^3)

h : effective height of the dam (m)

$\tan \alpha$: original streambed slope

$\tan \beta$: new streambed slope

ℓ : average length of the sedimentation solid

b : average width of the sedimentation solid

equation $A'B = \ell \tan \alpha$, $AA' = \ell \tan \beta$

it becomes:

equation $AB = A'B - AA' = \ell (\tan \alpha - \tan \beta) = h$

Therefore, it becomes:

equation (5-21) $\ell = \dfrac{h}{\tan \alpha - \tan \beta}$ (4-21)

Because:

equation $\Delta ABC = \dfrac{1}{2} h \ell = \dfrac{1}{2} \left(\dfrac{h^2}{\tan \alpha - \tan \beta}\right)$

It becomes:

equation (5-22) $V = \dfrac{1}{2} \left(\dfrac{bh^2}{\tan \alpha - \tan \beta}\right)$ (4-22)

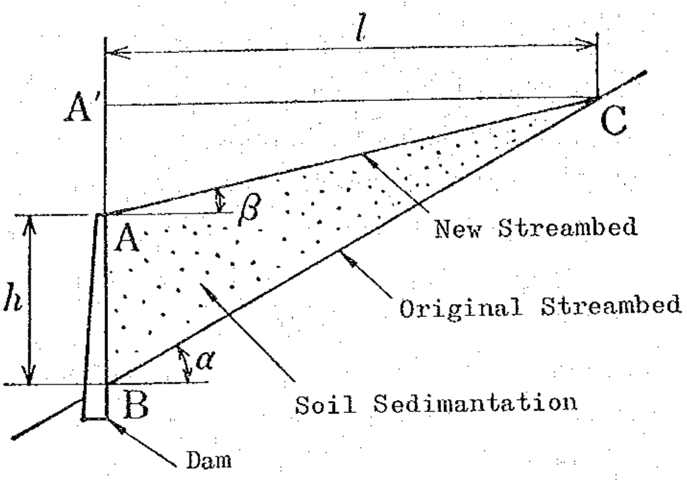


Diagram 4-19 Sedimentation Cross Sectional View

4. Spillway (Overflow Section) and Wings

I. Spillway

The part of the dam through which the overflow passes is called the spillway (overflow section) and its location, form, size, and sleeve structure is important for the preservation of dams.

(1) Location

The location of the overflow section is to be determined with the soil nature and topographical conditions of the both sides and the front apron of the projected construction site taken into consideration.

1) The overflow section may be provided anywhere when the stream banks and the streambed at the downstream toe of slope is comprised of rigid bedrock.

2) The overflow section is located in line of maximum depth when the stream bank are fragile without rigid rockbeds. (Diagram 4-20(a))

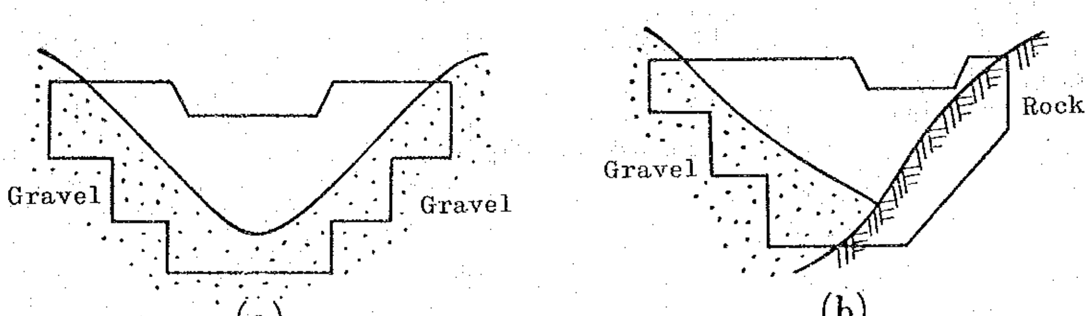
3) The overflow section is located closer to the side with the rockbed when the rigid rockbed exists only on one side. (Diagram 4-20(b))

4) The overflow section is located in a position where it will not cause erosions when structures such as revetment works, residential land, and arable land exist along the streambank downstream.

5) The overflow section is to be located in a position where are free from the effects by the water flow when collapse areas exist in the stream banks and hillsides upstream the dam. (Diagram 4-20(c))

(2) Shape

Trapezoidal, arch, and rectangular shapes are seen in overflow section. However, trapezoidal shaped overflow section are most commonly seen. The width of the overflow section should be increased as much as possible to reduce the overflow depth in order to decrease falling hydropower to minimize the washout toe of slope of the dam downstream. Furthermore, in streams with wide streambeds and large gravel discharge volumes, compound sectional shapes are employed at times as illustrated in Diagram 4-20(d) since gravel may sediment on the overflow section and increase turbulent flows.



(a) (b)

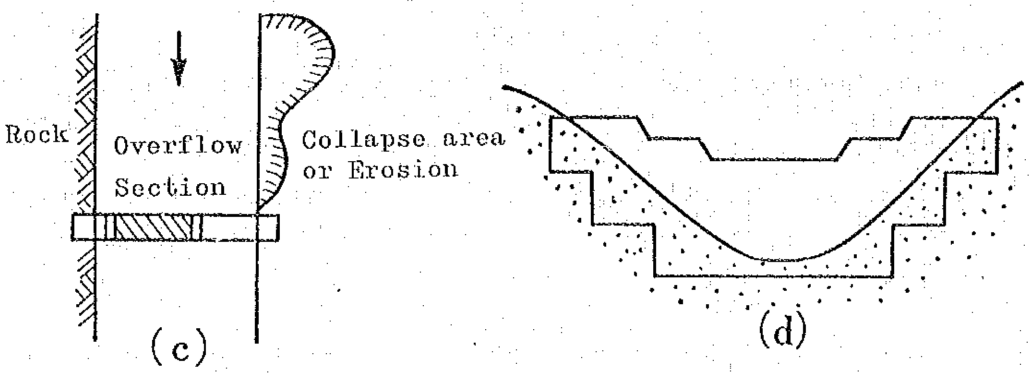


Diagram 4-20 Location and Shape of Spillways

(3) Cross Section

The dimensions of overflow section generally should be sufficient enough for running off the maximum discharge during floods safely with extra space for mud flow and driftwood taken in consideration. The maximum flood discharge is determined by the watershed area, precipitation, hillside gradient, forest conditions, and the hillside devastation conditions.

A sample method for determination of the cross section of the overflow section in case of open channel is explained as follows:

1) The maximum flood discharge Q is by Rational Formula (equation 4-19).

2) The diameter of the maximum gravel intra-streambed is selected. Generally, the average diameter of the largest sized intra-streambed gravel thought to have been washed down is applied.

3) The critical velocity vg of the maximum gravel is derived. $vg = K\sqrt{b}$ (Equation 4-14) is to be used.

4) The design accumulating gradient I is estimated. It should be about one half of the present existing streambed gradient.

5) A hypothetical overflow section is determined. The following calculations are to be performed using the values of 1) through 4).

(a) The cross section area F is derived by the following equation:

equation $$F = \frac{Q}{Vg} \qquad (4\text{-}23)$$

(b) The hydraulic radius R is derived by the following equation:

equation $$R = \frac{\alpha Vg^2 \pm \sqrt{(\alpha Vg^2)^2 + 4\ I \beta Vg^2}}{2I} \quad (4\text{-}24)$$

However, α and β are the coefficient of roughness of the Bazin's old Formula. This formula uses the Bazin's old Formula in Equation (4-15) C to consolidate the quadratic equation of R for derivation by the root formula of the quadratic equation:

equation $$\frac{1}{\alpha + \frac{\beta}{R}} RI = Vg^2 \qquad \frac{R}{\alpha R + \beta} RI = Vg^2$$

$$\rightarrow\ IR^2 \ \rightarrow\ Vg^2 R - \beta Vg^2 = 0$$

The wetted perimeter P is derived by the following equation:

equation $$P = \frac{F}{R} \quad (4\text{-}25)$$

Based on the cross section area F, hydraulic radius R, and wetted perimeter P derived, the cross section is approximated with extra room for the field site conditions taken in consideration.

6) Determination of the Overflow Cross Section

The following values at the estimated overflow section are to be derived.

(a) The cross section area F', Wetted Perimeter P', and hydraulic radius R' are to be derived by Equation (4-6).

(b) The safety factor n of the cross section area is derived by the following equation:

equation $$n = \frac{F'}{F} \quad (4\text{-}26)$$

(c) The mean velocity v' is derived by Equations (4-11/12) and the discharge Q' is derived by Equation (4-8).

(d) The safety factor of the discharge n' is derived by the following equation:

equation $$n' = \frac{Q'}{Q} \quad (4\text{-}27)$$

(e) The estimated overflow section is selected for use when the safety factors n and n' are about two to five.
Reestimate and recalculate for the selection of the overflow section when the derived safety factor is insufficient.

(4) Protection of Overflow Section

The overflow section is worn by the run-off gravel and may become destroyed by the impact of boulders or by the pressure from the flowing water. Protection structures are therefore executed as required.
The following describes the various protection work methods available.

1) Stone Consolidation
Closely consolidated rigid stone is used not only in wet masonry dams but also for the overflow section crown of concrete dams.

2) Rich Concrete Mixture
The crown is especially constructed by using rich mix concrete.

3) Other Methods
Other methods which use steel plates, old rails, and fibre glass boards on the spillway crown are also available.

II. Wing

The overflow section is sufficiently to pass the maximum flood discharge. However, it must be solidly constructed since mudflow and driftwood may be generated more than expected and overflow the sleeve. Furthermore, the crown of the dam wing in locations described in the following is to be provided with an upward inclination towards the stream banks. (An upward gradient must be provided:)
(i) immediately below hillside landslide areas, (ii) at mudflow generated areas, (iii) at driftwood run-off areas, and (iv) at curved areas of the stream.
The inclination in these cases is generally about the same as the design accumulating gradient of the dam. The required embedment depth of the wing into the stream banks is about 1 to 2 meters for bedrock and about 2 to 3 meters for soil and sand. Additionally, the connecting member of the wing to the stream banks often is damaged, resulting in the destruction of the dam, and therefore must be reinforced by protection works.

5. Dam Sections and Conditions of Stability

(1) Determination of the Dam Section

The selection of the dam section for linear gravity dams is generally accomplished by selecting the height, crown width, downstream slope, and then the upstream slope to attain a cross section satisfying the safety requirements. The dam base is determined along with these items.

1) Height

The determination of the dam height is made based on Section 3-(2) "Dam Height" described earlier.

2) Crown Width

The crown width is determined by the run-off gravel dimensions, overflowing water depth, and the design accumulating gradient upstream. The crown width is about 1.5 meters in ordinary devastated streams, over 2 meters in locations where large boulders and excessive mudflow are expected, and about 1 meter in small streams with smaller sized gravel run-off such as SHIRASU or ash.

3) Downstream Slope

The downstream slope must be steep in order to protect the slope surface from damage by the discharge and gravel, etc. overflowing the dam fall directly on the front apron. The slope is about slope ratio of 1:0.2 generally, and about slope ratio of 1:0.3 for lower dams under 6 meter height dam.

4) Upstream Slope

The upstream slope is determined in accordance with the dam height, crown width, and downstream slope determined earlier, satisfying the stability conditions for gravity dams.

5) Dam Base Width

The dam base width may be derived by the following equation when the height, crown width, upstream and downstream slopes are determined.

$$B = b + (n + m) h \qquad (4\text{-}28)$$

where
- B: Dam base width
- b: Crown width (1:n)
- 1:n: Downstream slope
- 1:m: Upstream slope
- h: Dam height

(2) Stability Conditions

The gravity dam resists various external forces with the weight of the dam body. These external forces may be segregated into the components of hydraulic pressure and mud flow impact, sedimented sand pressure, and earthquake force. However, only the hydraulic pressure is often considered with the unit weight of the water estimated between 1.2 to 1.8 t/m^3. The following conditions must be satisfied in order to ensure the safety of the dam against these external forces.

1) Not fall
2) The dam body has not broken.
3) The baseground has not broken.
4) Not slideable

These conditions are considered satisfactorily for a unit length of the dam.

1) Stability Against Falling

The action line of the resultant force of external force and the dead weight of the dam body must be through the dam base to prevent the dam from falling forward by external forces. Only the hydraulic pressure is to be considered as the external force (overflowing water depth is to be disregarded). The stability check diagram method is described in the following paragraph. The cross section of the dam is taken as ABCD as illustrated in Diagram 4-22.

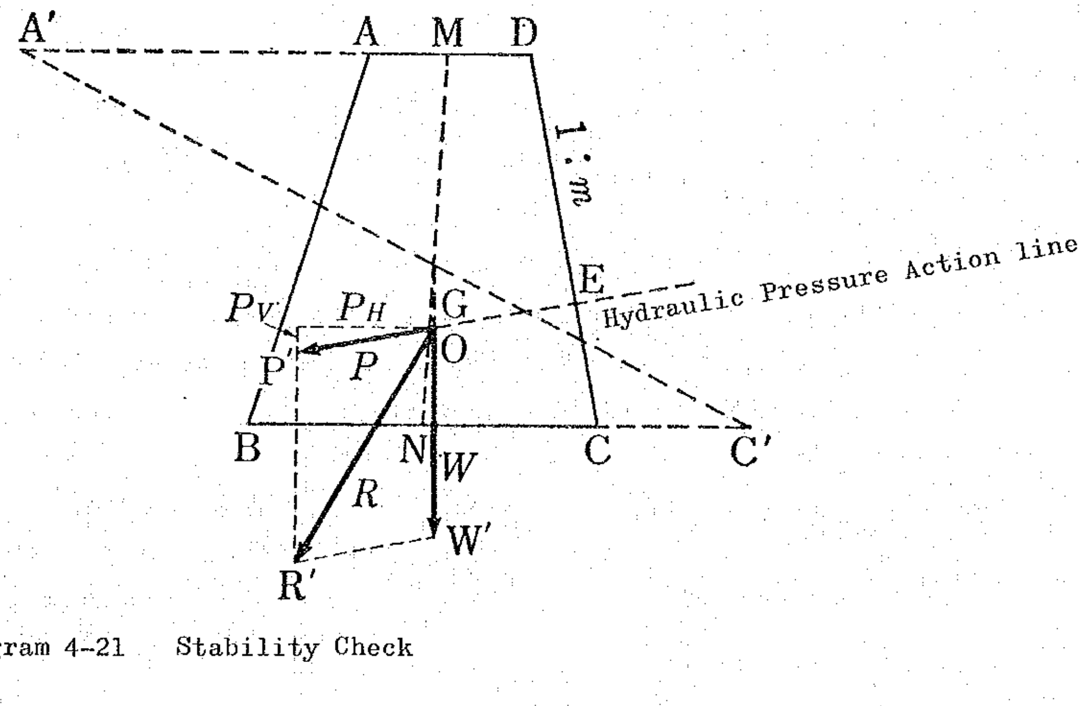


Diagram 4-21 Stability Check

With the length of the dam top AD as b, the length of the dam base BC as B, the dam height as h, the slope of the DC plane as 1:*m*, and the unit weight of the flowing water as γw, the overall hydraulic pressure P acting the DC plane per unit length may be derived by the following equation from the Equation (4-5).

(equation) $$P = \frac{1}{2} \gamma w h^2 \sqrt{1 + m^2} \qquad (4\text{-}29)$$

This overall hydraulic pressure acts the DC plane perpendicularly, and its acting point is at point E, 1/3h above the dam base. With the unit weight of the dam body as γm, the dam dead weight per unit length becomes:

(equation) $$W = \frac{1}{2} \gamma m (b + B) h \qquad (4\text{-}30)$$

and acts in a vertical direction through the center of figure G. Assume A' and C' are connected from the extensions of dam top AD and dam base BC so that it becomes AA' = BC and CC' = AD in order to derive the center of figure G of the trapezoidal section ABCD. Furthermore, assume that M and N lie on the median of dam top AD and dam base BC. Therefore, the intersecting point G of the line segments A'C' and MN is the center of figure of the trapezoidal section.

Draw the acting line of the overall hydraulic pressure acting the DC plane perpendicularly through point E and the acting line of the dead weight of the dam vertically from G.

They intersect at point O in order to derive resultant force of the overall hydraulic pressure P and the dead weight of the dam W. Establish force P line on the extension from O to EO and force W line on the extension of GO, and then draw OP' line, OW' line by certain scale size. Furthermore, by drawing a parallelogram OP'R'W' with these two sides, the length of the diagonal line OR' is the size of the resultant force R of the overall hydraulic pressure P and the dead weight of the dam W. Therefore, it is safe from falling forward when the action line of the resultant force R crosses the dam base BC.

2) Stability check of the Dam Body Against Destruction

The dam body is broken either by tensile force on the dam body or by the dam body giving way to compressive force.

In order for the dam body to be free from tensile force, the resultant force of the dead weight and the hydraulic pressure must be through the portion between middle two points in which trisect the dam base. This trisecting point is called the core point and the portion between the middle two points is called middle third. Now, assuming that the perpendicular force V acts point F which is distance a away from B on the dam base BC as illustrated in Diagram 4-23, the compressive force distribution on the dam base here becomes the maximum compressive force P_B at point B and minimum compressive force P_C at point C. The compressive force varies linearly between these BC points. P_B and P_C are described by the following equations:

(equation)

$$\left.\begin{aligned} P_B &= \frac{2V}{B}\left(2 - \frac{3a}{B}\right) \\ P_C &= \frac{2V}{B}\left(\frac{3a}{B} - 1\right) \end{aligned}\right\} \qquad (4\text{-}31)$$

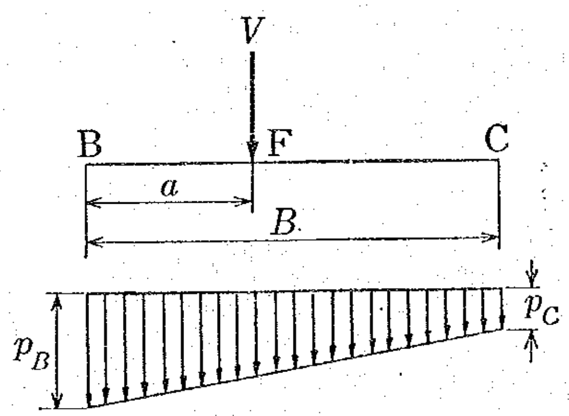


Diagram 4-23 Pressure Distribution on the Dam Base

In order for the tensile force to ineffective, $P_B > 0$ and $P_C > 0$. By analyzing this $2/3B > a > 1/3B$ appears, and therefore, it is proved that it is satisfactory when the perpendicular force V is through the middle third of the dam base.

In order for the dam body to be safe from destruction by compressive force, the allowable compressive strength K of the dam material must be greater than the maximum compressive force P_B on the dam body. Therefore, it is satisfied by the following equation:

equation $$K > \frac{2V}{B}\left(2 - \frac{3a}{B}\right) \qquad (4\text{-}32)$$

Allowable compressive strength of concrete is listed in Chart 4-4.

Chart 4-4 Allowable Compressive Strength of Concrete

Allowable Stress Variations	Allowable Strength		Summary
	t/m^2	kg/cm^2	
Allowable Compussive Strength	550	55	Age 28 days 1/4 of Compressive Strength
Allowable Bending Tensile Strength	30	3	Age 28 days 1/7 of Tensile Strength
Allowable Bearing Stress	600	60	Age 28 days 3/10 of Compressive Strength

3) Stability Check for Base Ground Destruction

It is satisfactory when the allowable bearing power of the base ground is greater than the maximum compressive force acting on the base ground. Land subsidence and destruction occur when the base ground allowable bearing power is insufficient. Additionally, since the maximum compressive force on the dam base acts the base ground directly, K may be substituted in Equation (4-32) as the allowable bearing power of the base ground.

4) Stability Check Against Sliding

The force acting to slide the dam body is the horizontal component P_H of the external force P. The force resisting the aforementioned is the reacting force by friction generated between the dam base and the base ground. The magnitude of the reacting force by friction may be derived by multiplying resultant force of the dead weight of the dam body W and the perpendicular component P_V of the external force by the coefficient of friction f of the friction between the dam body and the base ground. Therefore, the reacting force by friction must be greater than the force acting to slide the dam body in order to maintain the safe conditions. The following describes the this as an equation.

(equation) $f\ (P_V + W) > P_H$

$$f > \frac{P_H}{P_V + W} \qquad (4\text{-}33)$$

It may also be derived from the diagram since $P_V = \frac{m}{\sqrt{1 + m^2}} P$ and $P_H = \frac{1}{\sqrt{1 + m^2}} P$ on Diagram 4-22. Additionally, Chart 4-6 lists the coefficient of friction between the dam body and the base ground.

Chart 4-5 Lists the allowable Bearing Power of the Individual Types of Base Ground

Type of Base Ground	Bearing Power t/m^2	Type of Base Ground	Bearing Power t/m^2
Hard Rock	200~300	Normal Soil	3~10
Granite (Single Horizontal Bed Thicker Than 3m)	350~1000	Clay	5~20
Porphyrite (Single Horizontal Bed Thicker Than 3m)	300~350	Ballast Mixed Clay	5~30
Graywacke (Single Horizontal Bed Thicker Than 3m)	150~600	Ballast	30~60
Lime Stone (Single Horizontal Bed Thicker Than 3m)	160~240	Sand Mixed Ballast	20~50
Soft Rocks (Tuff, Sandstone, Shale)	70~150	Normal Sand	10~40
Sedimentary Rocks (Low Concretion)	50~60	Hard Clay	20~50
Conglomerate Gravel (High Concretion)	70~80	Wet Clay	15~20
Sand (High Concretion)	70~80	Mud	0
Sandy Clay	15~20		

Chart 4-6 Coefficient of Friction of the Individual Materials

Type of Material	Coefficient of Friction
Between Masonry and Masonry	0.6 ~ 0.7
Between Good Quality Stone and Masonry	0.6 ~ 0.7
Between Ballast and Masonry	0.5
Between Sand and Masonry	0.4
Between Dry Clay and Masonry	0.5
Between Wet Clay and Masonry	0.3

6. Drainage Hole

(1) Objective

The purpose of drainage hole is to shift the drainage and flowing during the construction of the dam and to reduce the hydraulic pressure on the dam body as well as upon the seepage pressure by sand sedimentation after completion.

(2) Location, Quantity and Dimensions of Drainage Hole

The location and quantity of drainage holes vary in accordance with the purpose of the drainage hole and the dam planning.
A single drainage hole above the streambed line is fufficient for dam construction on narrow streambeds. Several drainage holes must be provided when the streambed is wide since the main water stream shifts with every flood.
Several small drainage holes must be provided when the objective is to minimize the hydraulic and seepage pressure. In such a case, drainage holes of the bottom row are to be provided on the streambed line with the higher holes arranged in reeling form. The drainage holes here must be provided with adequate clearance with the neighboring hole and must be not perpendicular line. Additionally, drainage holes in the top row must be located 1.5 to 2.0 meters below the overflow section crown since they may become the cause of the dam destruction by the impact of the mud flow.

The drainage hole of the dam down stream is to be designed below the base ground of the dam upstream when constructing stepped dams. The dimensions of the drainage hole are to be sufficient enough for running off several floods for a year, since it becomes the weakest point of the dam body when the dimensions are excessive. Generally, drainage holes are square or rectangular with 0.2 to 0.4 meter sides but are also substituted with circular versions with similar sectional areas. Square or rectangular drainage holes with 0.5 to 1.0 meter sides are provided for tall dams executed for sand sedimentation.

7. Protecting from Scouring

It is desirable for the dam base ground to be comprised of solid bedrock. However, dams often must be constructed on gravel layers due to the objective of the dam. In such a case, the front apron is scoured by the flowing water and gravel overflowing the dam. It become a cause for the destruction of the dam when left in such condition for extended periods of time since the base ground also becomes scoured. Therefore, front apron work, counter dams, and riprap work are performed to prevent the scour. The selection of the these works must be made in accordance with the composition of the gravel in the streambed, discharge, and the dam height.

(1) Front Apron Work

This work is used to prevent the scour by securing the front apron with concrete in direct connection with the downstream part of the dam. This work is used in the downstream part of the stream when the diameters of the run-off gravel and streambed gravel are small with large discharge.

1) Length

Front apron works are executed in the part where the discharge overflowed dam and gravel fall. Therefore, the distance from the toe of downstream slope to the dam foundation level where the flowing water overflowing the dam overflow section down is derived by the following equation and is extended marginally in order to logically derive the length of the front apron.
(Diagram 4-25 refers. The front apron work height is to be at a level identical to the counter dam crown).

equation $$\ell = V\sqrt{\frac{2\ (h + t)}{g}} - nh \qquad (4\text{-}34)$$

where

ℓ : The length of the front apron from the toe of downstream slope (m)

h : The effective fall of the dam ① (m)

V : The surface velocity of the flowing water leaving the dam (m/s)

t : Overflow water depth (m)

g : Acceleration of gravity (9.8 m/S^2)

(1:n): Down stream slope

The length of the front apron is executed by experience as the following.
Incase of low dam, it is 2 times total length of overflow water depth and the effective fall of the dam.
In case of tall dam, it is 1.5 times total length of overflow water depth and the effective fall of the dam.

2) Thickness

The thickness of the front apron work has not yet been logically and actually analyzed. It is generally executed with a thickness of 0.5 to 1.0 meter by experience.

Front Apron Work is Generally Concrete Structured
Additionally, wire cylinders and wooden mattress are also used where the discharge is minimal and only when soil and sand are runoff. However, these lack durability and are only considered as temporary measures. Additionally, the gradient of the front apron is generally level, but is also sloped in accordance with streambed slope when the streambed slope is excessive.

4) Vertical and Side Walls

A vertical wall is provided at the tip of the front apron for depth of embedment since the downstream tip of the front apron on gravel layers becomes scoured.
The soft ground in the sides of the front apron becomes eroded

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① The height from the front apron to the overflow section crown.

② Frame constructed with logs and filled with boulders for sinking into the streambed.

since the flowing water falling from the dam generates turbulent flows on the front apron. Therefore, side walls are provided to protect the both sides. The height of the side walls must be determined so that the channel section of the front apron becomes larger than the overflow section of main dam. Additionally, the crown of the side wall must be provided with an upward gradient toward the upstream. The bottom part of the part connecting the side wall to the main dam must be far 0.5 to 1.0 meter from the shoulder line of the dam overflow section. This is done to prevent the side wall from destruction by the flowing water and gravel overflowing the overflow section of main dam.

(2) Counter Dam Work (Secondary Weir)

Low dams provided in the downstream of the dam to prevent the front apron from becoming scoured are called counter dams and the protected dam is called the main dam.

A water cushion is provided between these dams to minimize the impact of the falling water flow and gravel in order to prevent scouring. This work is executed in the upstream of torrents when the streambed is packed closely with boulders or when the diameter of the run-off gravel is large.

1) The Overlap Height of the Main and Counter Dams

The main and counter dam heights must be over lapped. To increase unnecessarily this overlapping height is not only uneconomical because of the increased height of the counter dam but also causes increased scour in the downstream of the counter dam. The overlap height generally must be about 1/3 to 1/4 the height of the main dam. (Diagram 4-25)

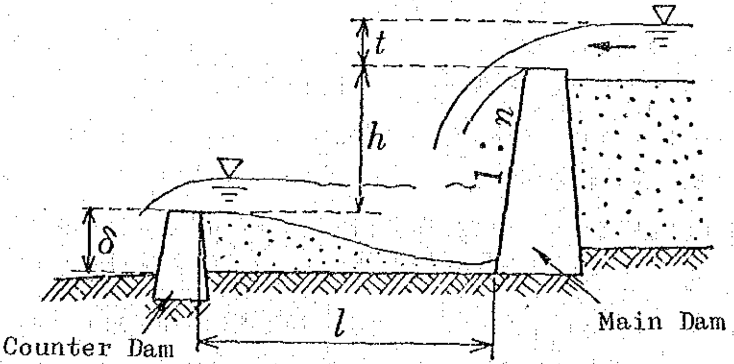

Diagram 4-25 Main and Counter Dams

2) The Interval between the Main and Counter Dams

The interval is determined in accordance with the calculated length of the apron.

(3) Joint Execution of the Front Apron and the Counter Dam

The front apron work and counter dam work are executed together when constructing a fairly tall dam in a stream with a large discharge and large sized gravel runoff. This is done in order to protect the front apron from abrasion and destruction by the boulders overflowing and falling from the main dam by the water cushion of the counter dam.

(4) Riprap Work

This is construction where concrete blocks and large boulders are dropped into the scouring part of the front apron in lieu of front apron works and counter dams to secure the these scouring part. However, the dropped materials often become washed away and therefore, this method should only be considered as a temporary measure.

8. Volume Calculations of Parts of a Dam

(1) Form and Cubic Volume Calculation

Calculations are made in the following order:

1) Draw the front view and plane figure of the dam with side views drawn at the sides and segregate the dam with horizontal lines intersecting the points where the shape of the dam changes on the front view and side view.

2) Derive the top length ℓ, bottom length L, and the height h of each section on the front view. Additionally, derive the top width b and the bottom width B of each section on the side view.

3) Derive the form area A of each section with the following equation:

(equation) $$A = \frac{\ell + L}{2} h \qquad (4\text{-}35)$$

When these are summed, it becomes the area of the surface of the dam when the dam slope is perpendicular. Additionally, the area when the slope ratio is 1:n is derived by multiplying $\sqrt{1+n^2}$ by the area in the perpendicular case. This area must be derived for both the up and downstream. Moreover, the areas of the form for the overflow section side and the drainage hole must be derived. The total sum of the aforementioned is the area necessary for the form of dam construction.

4) The individual dam section cubic volumes generally derived by the following equation:

(equation) $$V = \frac{h}{\ell}\left\{ 2\ (\ell b + LB) + \quad B + Lb \right\} \quad (4\text{-}36)$$

However, when the shape of the segregated section is rectangule in either one of the front or side view, the cubic volume is derived by the following equation:

(equation) $$V = A\,\frac{B + b}{2} \quad (4\text{-}37)$$

Additionally, when the shape of the regregated section is rectangule in both diagrams, the cubic volume is derived by the following equation:

(equation) $$V = AB \quad (4\text{-}38)$$

The cubic volume of the dam is the total of the above volumes minus the drainage hole volume.

(2) Base Excavation Volume Calculation

Calculations are made in the following order:

1) Draw the front view of the designed dam into the cross sectional diagram of the stream at the center line of the dam overflow section crown. Additionally, draw a side view at the side.

2) Segregate the front view with perpendicular lines. These lines are to be drawn where the shape of the dam base and the ground surface gradient change.

3) The following values must derived for the individual blocks on the front view.

   a) Height h': The height from the dam base to the ground surface at the center of each block.

   b) Extension ℓ: The length of each block.

4) The following values are to be derived on the side view in the center of each block.

   a) Add extra excavation space of the both sides to the dam base width for bottom width B'.

b) The top width b' may be derived by the following equation assuming that the slope of the base excavation is 1:n.

(equation) $b' = B' + 2nh'$ (4-39)

5) Derive the sectional area of each block by the following equation:

(equation) $$A' = \frac{B' + b'}{2} h' \qquad (4\text{-}40)$$

6) Derive the base excavation volume V' of the individual blocks by the following equation to sum them to derive the total base excavation volume of the dam.

(equation) $V' = A' \ell'$ (4-41)

9. Dam Construction

Lately, concrete dams are most widely constructed. The following explains the basic execution methods:

Construction Survey

The most important item for the construction of a dam is to expedite how efficiently, economically, and safely the work can be executed in the limited construction period. Therefore, it is highly important to establish an effective project schedule based on the survey data. The following listed surveys should be completed prior to designing the dam. However, further detailed surveys are necessary when constructing relatively large scale dams.

1) Topographic Survey

1/100 to 1/200 scale cross sectional diagram at the center of the dam is already completed at the design stage of the dam. Therefore, detailed surveys of the area in the vicinity of the projected dam construction site must be performed prior to the execution of the work. Exposed bedrock, spur line, flowing water level position, and large boulders, etc. are then inscribed onto the 1/500 to 1/1000 scale topographical chart of the projected dam construction site. Additionally, the temporary diversion channel, soil dumping area, and material storage area are to be indicated on the 1/500 to 1/1000 scale topographic chart of the surrounding area.

2) Geological Survey

Detailed surveys are required when constructing tall or arch type dams. Not only the bearing capacity of a gravel layer or the type of rock in bedrock is required to be identified but faults and fractured zones must also be thoroughly surveyed.

3) Weather and Flow Regime Survey

Throughly survey the weather and flow regime since they effect the work period most critically. Flood conditions and low water periods in particular are to be surveyed over with data for long periods of time. These data are highly beneficial for determining the execution period as well as designing the temporary coffer dam and temporary diversion channel.

4) Material Transportation Facility Survey

Most of the dam construction sites are located in areas where transportation is difficult. Therefore, the quality of the material transportation facility greatly effects the execution of the work. The best transportation facility is a road. Survey the width, gradient, curvature, and bridges as well as necessary improvements and repairs when an existing road exists.

Temporary Coffer Dam and Temporary Diversion Channel

The stream or river is blocked and is drained through the temporary diversion channel during the execution of the work in order to facilitate the work to attain certain dam execution results.
The best blocking and temporary diversion channel methods are selected in accordance with general judgement of the discharge, the topography, soil nature, thickness of the streambed sedimentation of the projected execution site, and the dimension of the dam. It is satisfactory with soil bag blockade in streams with minimal discharge. However, rigid soil and stone blockade, sheet pile blockade, and concrete blockade are employed in streams with large discharge.

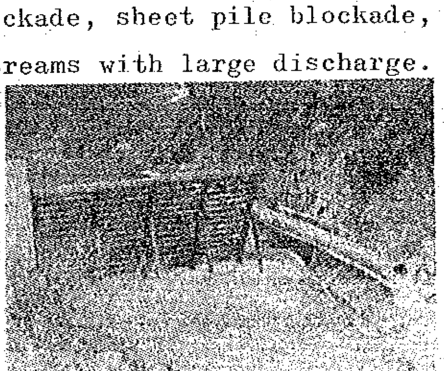

Diagram 4-26 Temporary Diversion Channel

The overall stream current is blocked in the up and downstream of the execution site and the water is drained with steel or wooden conduits or by a pump when discharge is minimal in a narrow width stream. Additionally, half of the stream is blocked at a time to execute the work when the discharge and the width of the stream is large. Furthermore, drainage tunnels are provided at times when the streambed width is narrow in contrast to the discharge and the dimension of the dam is large as well as the base excavation is deep.
The flow capacity volume of the temporary diversion channel is to be large enough to run off the flood discharge during the execution period based on the execution survey data.

(3) Base Excavation

The streambed is excavated down to expose the bedrock or the gravel layer is excavated to an adequate depth to secure the bed for the base when constructing the dam. This work is called the base excavation.

1) Gravel Base Excavation

The unfixed soil layer is removed and to excavate the thick gravel layer for 2 to 3 meters depth when the streambed is comprised of a gravel layer. The base excavation width is to be dug with 0.2 to 0.4m extra space than the dam base width for the execution of the form. Additionally, the base excavation gradient is to be safe enough against falling by rainfall and river bed water. The base excavation slope ratio is to be between 1:0.4 to 1:0.8 in case of soil. Mechanical excavation is recently more employed although manual excavation is also possible. However, unnecessary excavation volume often occurs when the entire excavation is accomplished by machinery.
Furthermore, re-excavation occurs in the streambed when it is left unattended for a prolonged period of time after the excavation. Therefore, the base excavation work is to be executed in combination with the concrete placing work schedule.

(2) Bedrock Excavation

Explosives are generally used to excavate bedrock. It is important at this point to pay close attention to prevent from loosening the bedrock in the vicinity of the dam foundation at this point.

Therefore, the bedrock is to be excavated manually or with a rock drill instead of explosives upon excavating down to the designed foundation height. The base excavation depth is to be determined in accordance with the structural quality of the bedrock and weathering conditions as well as the existence of faults and fractured zones. The depth generally is between 1.0 to 1.5 meter. The bedrock excavation slope is to be perpendicular or about ratio of 1:0.1. Supplementary excavation is not performed at times in case of a hard bedrock. Moreover, safety should be thoroughly maintained when employing explosives.

3) Residual Soil

The residual soil is to be primarily transported to the upstream of the dam wing with consideration given to preclude this soil from becoming washed out during floods.

Base Ground

1) Gravel Base Ground

The bottom is to be covered and packed with boulders, ballast and gravel prior to the dam construction when the dam supporting foundation is comprised of gravel.

2) Bedrock Base Ground

The bedrock surface is to be thoroughly cleaned with wire brush to make it rough, and is to be washed with water. Approximately 2 cm thick mortar is to be applied on this surface in order to erect the dam above this foundation.

3) Base Ground Treatment

Ground improvement treatment is to be applied in order to prevent ground subsidence and devastation when the base ground is fragile and soft. Piles are to be driven as pile foundation in case of a gravel foundation.

Additionally, the grouting method where mortar, etc. are injected by pressure is used in case of a bedrock.

Construction

1) Required Concrete Characteristics

The following characteristics are required for the concrete used for dam construction:

(a) Compressive Strength and Water Cement Ratio

The compressive strength required for concrete dams is generally said to be about 10 kg/cm^2. The necessary water cement ratio derived from this compressive strength is considerably greater than that based on durability of concrete. Therefore, the water/cement ratio is generally derived from durability of concrete.

(b) Durability and Watertightness

Durability is highly important for dams constructed in violent climatic variation areas due to the excessive surface weathering and erosion generations. The watertightness and durability are generally improved with greater unit cement volume and lower unit water volume. Air entraining agent is used for improving the durability and watertightness with approximately 3% air volume expected to be adequate.

(c) Abrasion and Impact Resistance

The spillway crown, downstream slope, and front apron, etc. which receive abrasion and impact are applied with rich mixture concrete, special concrete mixture, or special treatment at times.

2) Mix Design

Ready mixed concrete is often used lately. In such case, the mix design conditions are to be specified when ordering upon selecting the ready mixed concrete factory. It is normal then for the concrete manufacturers to design the concrete mix proportion in accordance with the specified conditions. The following are the conditions of the mix design:

(a) Cement Classification

Normal Portland Cement, Moderate Heat Portland Cement, Portland Blast Furnace Slag Cement, etc.

(b) Maximum Size of Aggregate

Larger maximum size aggregate is better in the case of a well-graded aggregate. It is generally said that maximum size 150 mm aggregate is most suitable for the dam concrete. However, 40 to 80 mm is the limit for ready mixed concrete.

(c) Slump

Excessive bleeding occurs when large slump concrete is used although the workability may be facilitated. Therefore, minimum slump concrete with the workability still maintained is employed. It generally is about 5 cm at the concrete placing site.

(d) Specified Concrete Strength

It is said that the maximum compressive strength of a gravity type dam with a height of 15 meters is about 10 kg/cm^2 and about 40 to 55 kg/cm^2 for an arch type dam. The specified concrete strength therefore is the compressive strength determined in accordance with the dam height and the stream condition added with the relative safety factors.

(e) Water Cement Ratio

The water cement ratio must be altered in accordance with the dam construction location and the weather condition, etc. It generally is about 58 to 62%.

3) Concrete Placing

Wooden forms must be thoroughly wetted with water and steel forms must be coated with oil prior to the placement of concrete as preparation work. Additionally, the part where the concrete is to be placed must be spread with mortar.

One hour may have passed at times from the mixing when employing ready mixed concrete for placement. In such case, close attention must be paid to prevent the material from separating since the slump and air volume become reduced during transportation.

Cable buckets and portable vertical chutes are employed for placing the concrete. Although the slanting shute is frequently employed due to its ease of employment. However, it should only be employed when other methods are unemployable since the material tends to become separated with this method. In such case, the concrete should not be dropped freely from heights in excess of 1.5 meters.

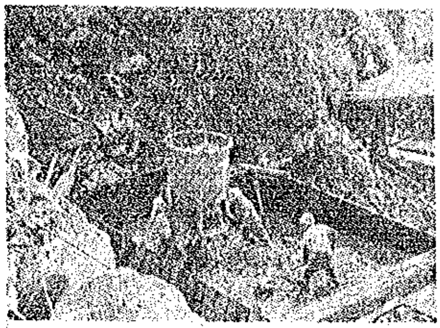

Diagram 4-27　Concrete Placing

Additionally, the concrete should be placed in the placement site only and should be avoided from movements of concrete. A layer of concrete placement should be about 30 to 50 cm in order to be able to be compacted as necessary, and the daily lift height should be between 0.75 to 2.0 meters.
The joint where the concrete is joined by the placement of fresh concrete over the solid concrete is called the horizontal placement joint. These horizontal placement joints should be avoided from being placed in points acted by excessive force as well as rear the water level where excessive weathering occurs since it tends to become the weakpoint of the concrete structures. The necessary concrete age for placing additional concrete is at least three days for lift heights under 1 m and five days for lift heights between 1.5 to 2.0 meters.
The concrete surface should be scrubbed with wire brush within 24 hours from hardening while applying water in order to thoroughly remove foreign objects and to make the surface roughness. The concrete is then placed after laying mortar on this prepared surface.
Crackes may become generated in large concrete structures by the expansion and contraction occuring during the hardening or the following temperature changing period or by the differential settlement of base ground. Flexible joints are therefore provided in order to preclude this crack generation. It is safer to provide these expansion joints every 15 meters for gravity dams with dam body lengths in excess of 30 meters.

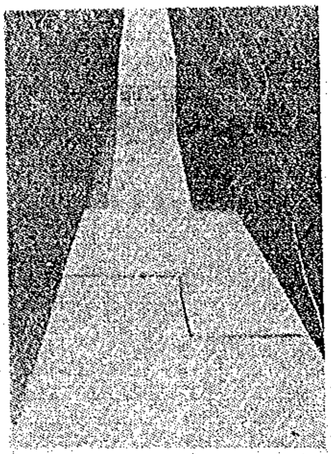

Diagram 4-28 Flexible Joints

The width of the joints are to be between 1 to 3 centimeters and is to be filled with joint materials such as asphalt.

4) Compaction

The concrete is to be compacted immediately after placing. When compacting with concrete vibrators, they are to be perpendicularly inserted with less than 60 cm interval. The concrete becomes free of cubic volume reduction when the compacting is sufficient, and water appears on the mortar surface as well as the concrete exhibits a uniformed blend appearance.

5) Curing

Although it is felt as though the work is completed when the concrete placement is finished. However, curing after the placement is also highly important since the quality of the curing during the solidifying process greatly effects the characteristics of the solidified concrete.

The adequate curing temperature is between 10 to 20° centigrade. Negative effects occur to the long term concrete strength and the strength may become reduced when cured at temperatures in excess of 35° centigrade. Additionally, concrete becomes safe against breezing once a strength of about 35 kg/cm^2 is attained although the concrete freezes at about -3° Celsius.

The concrete does not harden and the strength remains low as well as the durability and the water tightness become negatively effected when the concrete surface dries. Rice straw mats, etc. are therefore applied as well as water is sprinkled to maintain the surface damp in order to prevent the concrete surface from desiccating. Water sprinkling in this case should be accomplished from above the rice straw mats. The wooden forms should also be thoroughly sprinkled at this point. Moreover, water should be sprinkled as well as straw mats should be applied as deemed appropriate even after the form is removed. The curing time for normal Portland Cement and Moderate Heat Portland Cement is about 14 days and about 21 days for Portland Blast Furnace Slag Cement or when flyash cement is employed.

6) Gap-filling (MAZUME)

This is an important work to fill the gaps generated during the excavation of the base ground and the both sides stream banks. Both the base ground and the parts of wing embedment to stream banks are filled with concrete in case of a rockbed base ground. The base ground is filled with boulders and the parts of wing embedment to stream banks are filled along the original ground line with wet masonry retaining walls in case of gravel.

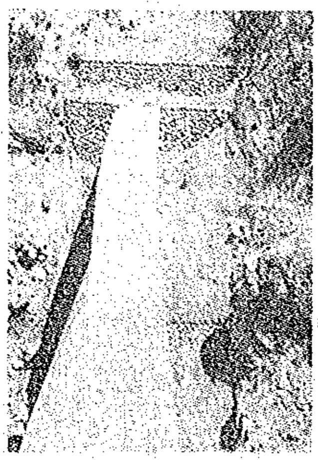

Diagram 4-29 Gap Filling

Section 4 Groundsel Sill Work

1. Objective

Since gravel discharge from the upstream is few, the streambed may become lowered or the foundations of retaining walls or other structures may become washed out in the downstream regions of streams where the upstream region is stable or where the restoration of devastation is progressed. Therefore, structures constructed across the stream to maintain the existing streambed from lowering as well as to prevent turbulent flows and to secure the streambed in the siltdeposit regions are called ground sill works.

2. Cross Section

The ground sill work is structurally almost identical to a soil conservation dam. Therefore, the ground sill work cross section is determined in accordance with the dam section calculation. The height generally is 2 to 3 meters, and is not to be excessively protruded on the streambed. The crown width is to be about 1.0 to 1.5 meters. Additionally, the upstream gradient is to be perpendicular and the downstream sloped is to be about ratio of 1:0.2.

3. Spillway and Sleeve

The position, shape, and section of the spillway are determined in accordance with those of the dam. The sleeves are to be thoroughly inserted into the banks in accordance with the hardness of the ground, the strength of the retaining wall and the topography since the ground sill work is often executed in the downstream regions of the stream.

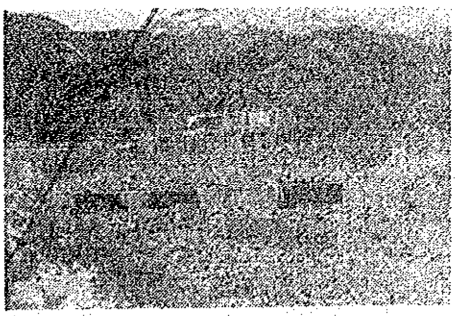

Diagram 4-30 Ground Sill Work

4. Foundation

The rooting of the ground sill work foundation is to be more than 1.0 meter when the streambed is comprised of a gravel layer, and about 1.0 meter when a rockbed or large boulders exist.

5. Wash-out Prevention

Aprons are to be provided when the streambed to be applied with bed sill work is comprised of a gravel layer. The length of the apron is to be two to three times the length of the head between the overflow and the apron surface. Additionally, the thickness of the apron is to be about 0.7 to 1.0 meter. Furthermore, the downstream edge is to be provided with a vertical wall as well as side walls are to be provided on the banks of the apron when rockbeds are not exposed.

6. Clearance

The streambed length that can be secured with a single bed sill work is short since the bed sill work is generally low. Therefore, bed sill works are often designed in steps. In such case, the clearance is to be designed appropriately in combination with the height and the designed streambed gradient. When assuming the height of the bed sill work to be constant, the clearance may be derived by substituting the of equation (4-21) as the distance.

Section 5 Revetment, Spur dyke and Channel Works

1. Revetment

(1) Objective

Not only is the streambed but also the banks are eroded in streams. Therefore, structures constructed along the stream on the banks in order to prevent the banks from side erosion are called retaining walls. Retaining walls are constructed in the concave sections of the banks where the discharge impact, land-creep hillsides or possible land-creep hillsides, and the fragile sections of the banks where the dam sleeves are inserted. Additionally, retaining walls constructed in the downstream are executed in combination with the ground sill work and spurdyke as the main body of the channel work in order to prevent the banks from devastation.

(2) Variations

Retaining walls may be classified into concrete, concrete block, concrete frame, stone filled concrete, wet masonry, stone consolidation, cylinder, and wicker depending on the structural material employed. Concrete, concrete block, and wet masonry retaining walls among the aforementioned are most commonly executed. Additionally, cylinder and wicker retaining walls are executed at times in smaller streams with minimal gravel discharge.

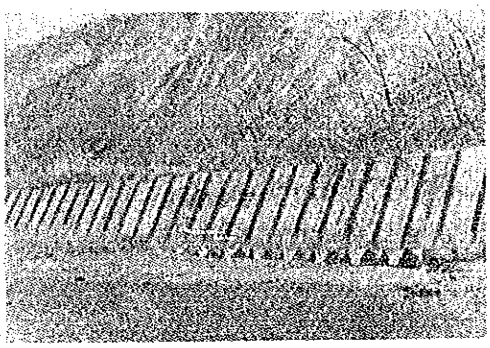

Diagram 4-31 Concrete Revetment works

Concrete retaining walls are executed in areas where the soil on the hillside may move and where ample safety is required due to the excessive dorsal pressure. The slope gradient is generally sufficient ratio of 1:0.3 to 1:0.5. Furthermore, although the thickness is determined in accordance with the soil pressure, slope gradient, and height, the average height is generally to be between 0.3 to 0.5 meter with the foundation broad and the crown narrow. Moreover, back filling gravel and weeping are necessary in order to minimize the dorsal water and soil pressure.

(3) Gradient Line

Curvatures are to be slow as much as the topography allows when determining the gradient ① line of reventment. Additionally, the

① The line of front slope

gradient line is to be isolated away from the land creeping hillfoot to provide sedimentation areas for the fallen earth when soil falling from the hillside is expected after the execution of revetment.
Both ends in the up and down stream of the revetments are to be well werapped into the banks to preclude the discharge entry to the rear from the edges.

(4) Height

The height of the revetment crown is to be 0.5 to 1.0 meter above the designed water level [1] so that the crown does not become overflown by the discharge during floods. The height in the convex parts of curved flow areas are to be sufficiently high for safety since the water level elevates considerably higher than the facing bank. When designin; revetment in the upstream of dams and ground sill wirks, they are to be erected in the some or higher heights as the crown of the wing of these structures. Additionally, the upstream parts are to be raised along the designed streambed gradient.

(5) Foundation

The flow velocity increases in the vicinity of the revetment, causing the foundation to become eroded when revetment are constructed. Therefore, the revetment foundations are buried at least one meter deeper than the designated streambed height. The foundation of the revetment at the upstream junction to the side work is to be about one meter below the overflow section crown and have the same height as the sidework foundation in the downstream, and it is to be retreated from immediately below the overflow section shoulder according to the side wall.
Additionally, foundation procedures such as pile driving, base logs, and stepped foundation, etc. are used when the foundation is soft.

(6) Scour Prevention

Rooting as well as hardening work are executed in areas in which revetment foundations are prone to become scoured by the discharge. Side works are most desirable for root hardening construction,

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① The designed water level is water level to be able to runoff with safety designed high water discharge in the stable river bed. Maximum flood discharge in the past is employed as the designed high water discharge.

however, rubble, wood work mattress, and various concrete blocks are used for partial scour prevention as well as for scour prevention in gentle streambed gradient areas.

2. Spur dyke (Grain works)

(1) Objective

Spur dykes are structures protruding towards the center of the flow from the banks and are constructed along with revetments, etc. Objectives of this work are to:

1 Isolate the current from the banks to prevent erosion of the banks.

2 Reduce the flow velocity in order to sediment soil.

3 Restrict the channel width to suppress turbulent flows and drifts to prevent side erosion.

This work is generally done in places with wide streambeds and gentle streambed gradient.

(2) Variations

Spur dykes are segregated into perpendicular, upward, and downward spur dyke depending on their direction of protrusion towards the center of the flow as illustrated in Diagram 4-32. These three types each posses exclusive features. Soil Sedimentation occurs between the spur dyke, and the scour of the head is minimal with perpendicular spur dyke. Soil sedimentation along the banks and the spur dyke as well as the scour at the head of the spur dyke is larger for the upward spur dyke. Soil sedimantation between the spur dyke and the scour of the head are minimal with the downward spur dyke. Additionally, these are further segregated into overflowing spur dykes depending if the discharge overflows the spur dykes. They may also be segretated in accordance with their structural material into concrete, concrete block, wet masonry, cylinder, and wood frame types, etc.

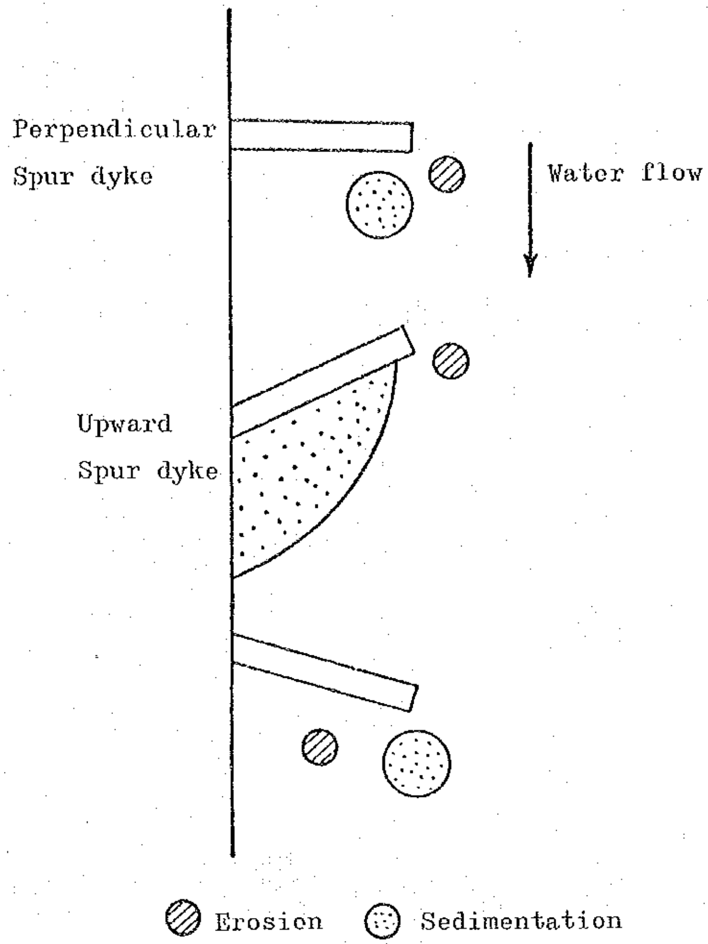

Diagram 4-32 Groine

(3) Type Selection

The type selection is generally performed as described next. In short streambank sliding areas in the upstream, downward and non-overflowing spur dykes are planned in the edge of the upstream of the land creeping area in order to isolate the current from the hillfoot to prevent the land creepage from expanding. Additionally, multiple perpendicular and non-overflowing spur dykes are planned when land creep areas exist for an extended length along the streambank. The erosion of the facing bank should also be considered since the planning is usually performed for one of the banks in land creeping areas.

Perpendicular and non-overflowing spur dykes are planned for the facing banks to face each other in soil sedimentation areas with wide streambeds where the movement of gravel and erosion of the bank are excessive by the turbulent flow and drift.

(4) Height

The height is generally designed so that the crown of the spur dyke does not become overflown during floods. Additionally, the crown is to be provided with 1/10 to 1/15 gradient from the bank towards the center of the flow. Furthermore, the foundation of the tips are to be buried deep and long into the streambank in order to prevent erosion of the head.

(5) Length & Clearance

The length of spur dyke is to be determined upon through investigation of the channel and discharge conditions. Generally, it is designed short for wildstreams. Spur dykes are often provided continuously, and the clearance is to be determined with the length and direction of the spur dyke, the strength of the current and the streamed gradient taken into consideration. The clearance may be extended if the length of the groyne is long. The clearance is generally 1.5 to 2 times the length of the spur dyke. The clearance is reduced in the subsided bank of the curvature and is widened in the protruded bank.

3. Channel Works

(1) Objective

Turbulent flows become generated during floods causing excessive side and longitudinal erosion and may cause disasters in the soil sedimentation area downstream. Therefore, the work to erect structures to protect the bank and to secure the streambed by providing a constant flow channel to such streams in order to prevent these disasters is called channel work.

(2) Design

The channel work may become buried or destroyed when the work is executed in the upstream in the devastated condition. Therefore, soil conservation works are executed upstream to reduce the gravel run-off and restore the devastated land prior to the execution of this work. Furthermore, when extensive time is needed for the restoration of the devastated land or when urgency is required for the restoration of disasters, the channel work is planned by erecting a dam for soil conservation wear upstream.

(3) Gradient

Maintain the gradient line as linear as possible when determining the gradient line of channel work. Attain the largest curvature radius possible in the event that it must be curved.

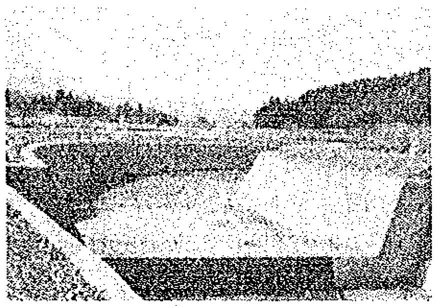

Diagram 4-33 Channel Work

Additionally, refrain from connecting curves with opposing directions and provide straight sections in between.
These factors must be thoroughly followed when the gradient is increased. The gradients of both banks must be parallel in order to avoid the rapid change of streamwidth. Furthermore, in the junction point of the main and tributary streams, the center line of these streams must be intercepted at a small angle since turbulent follows may be generated by the impact of the adjoining currents resulting in the devastation of the banks when this angle is large.

(4) Design Accumulating Gradient

Stable gradient are used for the design accumulating gradient to prevent soil sedimentation as well as erosion of the streambed upon completion of the channel work. This design accumulating gradient is designed generally at about 1/2 the original streambed gradient. The gradient may be constant from the beginning to the end of the channel work when the execution distance is short. However, the gradient must be varied where the upstream gradient is steep and becomes gentle towards the downstream when the construction distance is long. The difference between the existing and the design accumulating gradient is to be corrected with the head of the ground sill work. Multiple ground sill works with smaller heads should be designed in stead of a few large headed ground sill works at this point.

(5) Work Types

Sill bed work as well as triple side channel work, etc. are done as required by the condition of the stream along with the erection of revetment during channel work.

1) Stone Removal & Excavation

Large boulders on devastated streambeds are to be removed to determine the channel when executing channel work in accordance with the designed grading line. Additionally, the streambed soil must be excavated at times to form to designed channel section. This is done because it is safer to lower the riverbed than to erect embankments in channel work.

2) Ground Sill Work and Bed Gindle

Ground sill works are executed (along with bed gindles at times) the protection of the revetment as well as to prevent streambed erosion when executing channel work.

The wing of the ground sill work is to be sufficiently inserted into the ground in order to border the channel works.

This is done is order to prevent the damage to the downstream of the ground sill work in the event that a part of the channel work is destroyed. Additionally, in areas proved to large volume of subsurface water, in case that the destruction of the channel work may occure, drainage closed conduits are provided in addition to the ground sill work.

Bed Gindles are done for the purpose of fixation of original streambed and the reinforcement of retaining wall when the distance of the ground sill work is long, and may be thought of as headless ground will work. Therefore, the corwn of this work is to be aligned with the streambed during design.

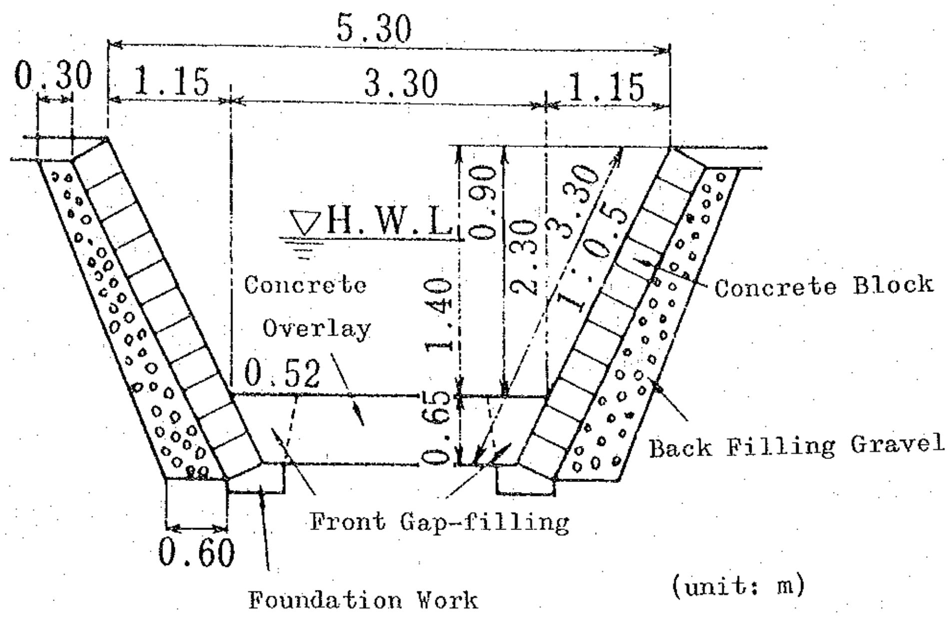

Diagram 4-34　Channel Work (Concrete Block)

3) Triple Side Pave Channel Work

The scouring advances when the streambed starts to settle and there is a danger of revetment destruction in narrow channel streams where the revetment foundations are almost in contact with each other. In such torrents, the streambed is secured with stone and concrete along with the construction of revetments. This is called the triple side paved channel work.

CHAPTER V LANDSLIDE (LANDCREEP) PREVENTION

SECTION 1 Landslide Investigation

Actual landslide conditions cannot be readily observed from the ground surface visually due to the cause being the generation of the sliding surface underground. Therefore, in order to achieve efficient landcreep prevention, the cause, extent, and movement conditions must be studied with the methods described in the following.

Landcreep investigations may be segregated into collection of existing date, weather research, topographic survey, geological survey, ground surface shift survey, sliding surface survey and ground water survey.

1. Existing Data Collection

Landslide occurence area often have a history of land movement, since the land in such areas are moved continuously or intermittently over extended periods of time. Therefore, it is necessary to primarily study the generation dates, extent, cause, and movement speeds through local tales, etc. in order to clarify the overall landslide.

2. Weather Research

Rainfall, earthquakes, and removed of slope bottoms are some of the known causes of landslides. However, it is also know that landslides frequent occur after rainfalls. This is due to high correlation between landslide shift and precipitation. Therefore, investigation of the precipitation is considered to be most important of all weather research aspects.

3. Topographic Survey

The extent and range of the landcreep may be observed on 1/25,000 or 1/50,000 scale topographical maps with exclusive creeping land features such as ununiformed contour lines, as well as increased numbers of <u>SENMAIDA</u> (thousand terraced fields) as illustrated in diagram 5-1, swamps, and ponds. These features are also helpful in location and detection of landcreep areas where potential landcreep phenomena are