A DESIGN REPORT
OF
ONDOLOKO SPATE IRRIGATION
PROJECT

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FARM-AFRICA
August, 2001
Konso, SNNPRS
# Table of contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acronyms</td>
<td>3</td>
</tr>
<tr>
<td>Acknowledgement</td>
<td>4</td>
</tr>
<tr>
<td>1. Introduction</td>
<td>5</td>
</tr>
<tr>
<td>1.1 Location of the project area</td>
<td>5</td>
</tr>
<tr>
<td>1.2 Review of existing farmers practice in Konso</td>
<td>5</td>
</tr>
<tr>
<td>1.3 Review of existing conditions in Ondeloko canal in particular</td>
<td>6</td>
</tr>
<tr>
<td>2. Objectives of the proposal</td>
<td>7</td>
</tr>
<tr>
<td>3. General geology of the project area</td>
<td>7</td>
</tr>
<tr>
<td>3.1 Geology at head work site</td>
<td>7</td>
</tr>
<tr>
<td>3.2 Geology at the main canal</td>
<td>8</td>
</tr>
<tr>
<td>3.3 Geology at farm lands</td>
<td>8</td>
</tr>
<tr>
<td>4. Maximum floodwater estimation and hydraulics of flow</td>
<td>9</td>
</tr>
<tr>
<td>5. Lay out of the scheme</td>
<td>16</td>
</tr>
<tr>
<td>6. River training works</td>
<td>17</td>
</tr>
<tr>
<td>7. Infrastructures in the project</td>
<td>17</td>
</tr>
<tr>
<td>7.1 Weir</td>
<td>17</td>
</tr>
<tr>
<td>7.2 Spill way</td>
<td>18</td>
</tr>
<tr>
<td>7.3 Out lets</td>
<td>19</td>
</tr>
<tr>
<td>8. Crop water requirement and canal capacity</td>
<td>19</td>
</tr>
<tr>
<td>9. B.Q and project cost</td>
<td>23</td>
</tr>
<tr>
<td>10. Operation and maintenance</td>
<td>26</td>
</tr>
</tbody>
</table>
Acronyms

Q  Flood discharge (m$^3$/s)
q  Flood intensely (m$^3$/s/m)
p  Perimeter (m)
A  x-area (m$^2$)
R  Scour depth, mean hydraulic radius (m)
S  Slope
V  Velocity (m/s)
n  (Manning’s roughness coefficient)
u/s up stream
d/s down stream
TEL Total energy level (m)
WL water level
f silt factor co-efficient
He Head above weir crust (m)
G Specific gravity
g gravitational acceleration (m/s$^2$)
M Catchment area (Km$^2$)
F Froud No
Acknowledgement

I would like to express my heartiest gratitude to Ato Abay Gulema Who suggested and commented on the possible design alternatives from his long year rich experiences. The valuable publications he forwarded me have also given me a very good insight for this project.
Introduction

1.1 Location of the project area

The project area is found in Konso Special Woreda, SNNPRS

Ondoloko spate irrigation canal is one of the six traditional canals branching both to the right and to left of the seasonal river Kapemage. The Kapamage River flows in a general direction of west to east from the ridges of Be-ayde and Gojabo Kebele down to join the Yanda River. Specifically the Ondoloko spate canal is located in river Kapamaga about 16 km from Karat, capital of Konso special Woreda crossing the main road to Arbaminch. The Ondoloko canal is located 2km of the road up stream from the main road branching to the right of the river flow. Geographically the site is located 37° 25’ longitude and 5° 28’ latitude on 1:50:000 scale top map.

1.2 Review of existing farmers spate irrigation practice in Konso

The Konso people have a long history of effective utilization of the rainfall that falls on their land by trapping, diverting and impounding runoff through elaborate systems of terracing and channelling.

The traditional diversion techniques used include building of soil bunds from the riverbed materials reinforced with brushwood and stones. These structures can divert proportions of flood into the canals. The positioning, height and width of these structures is a matter of sound judgement and considerable experiences at each particular diversion site.
The main problems the farmers faced is that the spurs are partially or completely washed away by unmanageable flood flows which pass on down stream to spurs lower down to down streams where it is diverted to other command areas. The spurs are washed away before the total command area has been irrigated and mostly water cannot be diverted to the fields again until the spur has been rebuilt. How soon this can be achieved will depend on the time interval to subsequent river flows and the availability of enough labour force. Thus, the bunds cannot always be rebuilt before the ensuing flood.

It is worth to mention that traditional systems are relatively cheap to build but require considerable repairs to remain operative. If small to medium flows arrive, the spur can be effective, but medium to large floods can result in the expenditure of much labour with very little benefit as all the spars may be swept away the available flood is wasted. Thus, the probability of irrigation of total command area of each traditional diversion is variable and risk prone.

The main canals are deep and narrow and have steep bed slopes. Thus, the high river flows often produce scouring velocities in the canals, which are quite long become more incised, there by causing some fields to go out of irrigable command.

Water rights are determined by the communities and seem to provide fair and equitable distribution and usage of available water supplies to all participating farmers. The construction and rebuilding of spurs and distribution of flows through the canals are communal activities organised by the farmers.

1.3 Review of the specific condition of Ondoloko canal

The Kapamage River flows with a number of meanders till it joins the Yanda. From this river about 6 traditional canals are branching both to the rights and left sides of the river. The 3 canals are found up stream of the Ondoloko and the other two are at the lower reaches below the main road crossing the river. Ondoloko branches to the right. In this
canal about 190 farms from Tishmale Kebele are benefitting. The average dimensions of the existing canal are described as below.

Average bed width = 1.76m  
Average top width = 4.33m  
General slope of the canal = 0.008  
Total length of the canal up to the command = 742m

The riverbed in dominantly a deep deposit of course sand mixed with shingles. This deposit of sand is resulted due to frequent meandering of the river and relatively gentle slop of the riverbed. These two factors undoubtedly reduce the velocity of flow creating a good condition for the river to dump its bed materials.

And the fine deposit of silt at the entrance of the canal show that there is an obstacle in the free passage of water through the canal and the siltation of the canal bed is frequent. And this canal is a good example of all the canals in the Woreda facing the major problems discussed in the review of the traditional canals in the Woreda above.

2. Objectives of the scheme

The main object of this proposal is to increase the household income of the beneficiary farmers by effective utilization of the available scarce floodwater resource. There by minimizing the vulnerability of poor farmers to the recurrent droughts in the Woreda.

The immediate objectives will therefore be assisting the farmers in their effort to divert the floodwater to their farmyards by constructing stable structures that will reduce wastages in labour, material and the core scarce resource water.

3. General geology of the project area

3.1 Geology of head work site
In order to put an appropriate diversion structure to withstand the forces of water, the nature of the riverbed has to be studied. Therefore a sample pit on the proposed weir axis was excavated to a depth of 3m. The excavated soil sample depicts that the riverbed is deposit of course sand mixed with shingles and cobbles. But a sample pit on the proposed spillway point shows different layers of soil ranging from course sand to silty clay in the lower layers. Therefore it is possible to put a stable masonry structure for the sluice get as the point is out of the main river course.

3.2 Geology of the main canal

The canal bed in the upper reaches is dominated by sand deposits but both banks show alternative layers of different soil textures the canal bank is bare from vegetation cover. We can say that the perimeter of the canal is rough dominated by silty loam soils. Such kinds if soils have a Manning roughness coefficient of \( n = 0.028 \).

3.3 Geology of Farm land (texture and water holding capacity)

Beneficiary farmers categorised their farmland into three classes.

1. Soils that need one irrigation for the crops to mature during the whole growing periods of the crop.

2. Soils that need two irrigations for the crops to mature during the whole growing periods of the crop.

3. Soils that need three irrigations for the crops to mature.

The majority of their land needs 3 irrigations during the growing periods of the crop. Soil samples, collected from the three categories of the soil, indicate that soils that need 1-irrigation are silty clay, soils that need 2 irrigations are silty loam and soils that need 3
watering are silty soils with small amount of sand content. (Soils are differentiated by feel method.)

4. **Maximum flood water estimation**

The maximum floodwater passing through the river at the proposed diversion site could be calculated by using different empirical formulate such as Ingels' and Dicken's formula. But the accuracy of the value is dependent on the selection of the multiplying constant, which ranges from 2-40.

In Dicken's formula for example,

\[ Q = CM^{1/4} \]

From 1:50,000 topographic map, it is possible to delineate the catchment area, M, that is about 26km². In computing Q (maximum discharge) for C=2, Q=23m³/s and for C= 40, Q = 460m³/s. and the selection of C needs a thorough study of the whole catchment.

Therefore instead of computing the maximum flood from Empirical formulas, it is more reliable to calculate the maximum flood by deciding the maximum flood level by local enquiry with old village men, at a number of points on w's and d's of the proposed diversion site and by observing the flood marks on banks.

4.1 **Determination of river bed slope**
<table>
<thead>
<tr>
<th>No</th>
<th>Station No</th>
<th>Distance</th>
<th>Elv(^a).</th>
<th>Accumulative height</th>
<th>Total Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0+20</td>
<td>0</td>
<td>18.208</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>0+40</td>
<td>20</td>
<td>18.468</td>
<td>0.26</td>
<td>2.6</td>
</tr>
<tr>
<td>2</td>
<td>0+60</td>
<td>20</td>
<td>18.603</td>
<td>0.395</td>
<td>6.55</td>
</tr>
<tr>
<td>3</td>
<td>0+70</td>
<td>10</td>
<td>18.613</td>
<td>0.435</td>
<td>4.15</td>
</tr>
</tbody>
</table>

\[ H_{avg} = \frac{2A}{L} = \frac{2(13.3)}{50} = 0.532 \]

\[ I_{avg} = \frac{0.532}{50} = 0.01064 = 1.064\% \]

4.2 River X-section /flow X-section/ 60m U/S of the weir axis

Flow X-sectional area \((A)\)

\[ A = \frac{1}{2} (7+84) 0.8464 = 6.348 \text{m}^2. \]
\[ P = 2(0.989) + 7 = 8.96 \text{m} \]

\[ R = \frac{A}{P} = \frac{6.348}{8.96} = 0.708 \]

\[ V = \frac{1}{n} R^{2/3} S^{1/2} \text{ take } n = 0.025 \text{ (for unlined rough canal)} \]

\[ V = \frac{1}{0.025} (0.708)^{2/3} (0.0100)^{1/2} \]

\[ = \frac{0.025}{3.27 \text{ m/s}} \]

\[ Q \text{ (discharge in the river) during the maximum flood is calculated as follows} \]

\[ Q = V \cdot A \]

\[ = 3.27 \text{ m/s} \times 6.34 \text{m}^2 \]

\[ 20.75 \text{ m}^3/\text{s} \]

4.3 Determination of maximum scour depth (R)

To determine the maximum scour depth we have to find the water way, and

\[ P \text{ (water way)} = 4.75 \sqrt{Q} \]

\[ = 4.75 \sqrt{20.75} \]

\[ = 21.6 \text{m} \]

Let the effective waterway over the crest (B) be 12m. At weir axis 12 < 22.556 m the flow is contracted at the weir site.

\[ q = \frac{Q}{B} = \frac{20.75}{12} = 1.729 \text{ m}^3/\text{s/m} \]
For contracted water way we have to use Lacy's equation for determination of maximum scour depth below the down stream maximum flood level (maximum flood level before the weir in constructed.)

\[ R' = 1.35 \left( \frac{q^2}{f} \right)^{1/3} \]

\[ f = 1.5 \text{ (of course sand bed material)} \]

\[ R' = 1.35 \left( \frac{1.725^2}{1.5} \right)^{1/3} \]

= 1.698  to be safe and the weir axis at turning point lets take \( R = 2R' \)

\[ R = 2(1.698) = 3.38 \text{m below maximum flood water level.} \]

4.4 Over flow depth \((Hc)\)

\[ Q = C_L H \text{e}^{3/2} \]

\[ H_e = \frac{(20.75)^2}{3} = 1.01 \text{m} \]

\[ 1.75 \times 12 \]

\[ H_e = \text{Total energy head above crest level including velocity head.} \]

Up stream TEL

\[ = \text{Crest level} + H_e \]

\[ 1218.704 + 1.01 = 1219.714 \]

U/S energy depth \((Y_u)\)

U/S TEL – lowest river bed level in the weir axis cross section

\[ 1919.714 - 1217.866 = 1.848 \text{m} \]
\[ Y_0 = Y_1 + \frac{V_1^2}{2g} \quad Y_1 = \frac{q}{V_1} \quad Y_2 = \frac{q}{V_2} \]

\[ 1.848 = Y_1 + V_1^2 \quad V_1 Y_1 = 1.729 \]

*By trial and error method*

<table>
<thead>
<tr>
<th>( Y_1 (m) )</th>
<th>( V_1 (m/s) )</th>
<th>( Y_2 (m) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>8.645</td>
<td>4.8</td>
</tr>
<tr>
<td>0.3</td>
<td>5.763</td>
<td>2.2</td>
</tr>
<tr>
<td>0.35</td>
<td>3.94</td>
<td>1.856</td>
</tr>
<tr>
<td>0.351</td>
<td>4.926</td>
<td>1.848</td>
</tr>
<tr>
<td>0.4</td>
<td>4.3225</td>
<td>1.353</td>
</tr>
</tbody>
</table>

*Froude No calculation*

\[ F_1 = \sqrt{\frac{V_1}{9Y_1}} \quad \& \quad F_2 = \sqrt{\frac{V_2}{9Y_2}} \]

\[ F_1 = \frac{4.926}{\sqrt{(9.8)(0.351)}} = 2.654 \]

\( \Rightarrow \) Flow creates an oscillating jump and needs a stilling basin for dissipation of kinetic energy.

*Sequent depth, \( Y_2 \)*

\[ Y_2 = Y_1 \left( -1 + \sqrt{1 + 8F_1^2} \right) \frac{2}{2} \]
Location of the jump (if the fall is to be constructed in Ogee shape)

The jump occurs at:

\[ \text{D/s WL} - Y_2 \]
\[ 1219.02 - 1.154 \]
\[ 1217.866 \]

5. Layout of the scheme

In order to determine the height of the weir that should be raised from the riverbed, it was necessary to check the longitudinal profile of the canal up to the head reach of the command area. The surveying report shows that the riverbed at the diversion weir axis site is much higher than the command area. Therefore it is not important to raise the weir crest to irrigate more command. However, as discussed in the review of the existing system of the particular site, sedimentation of the silt at the upper reaches of the canal crates much trouble in blocking the flow of water at the entrance. Therefore to flush out the silt deposit in front of the weir adequate gradient should be given between the weir crest and the bed of sluice gate at the spillway point. Therefore the height of a minimum market available gabion size of 0.5m is raised for proper flushing of sediment loads in the approach canal and in front of the weir from the average riverbed level.

Measuring the crest length from a scale drawing of the river cross-section profile at 0.5m above the average riverbed shows that length of the weir is 12m (effective).

6. River training works

6.1 Marginal bunds

By raising the weir crest by 0.5m will definitely raise the maximum flood level out of the existing bank level. It was calculated that after construction of the weir the U/S WL would raise from 1219.144 (MFL before weir construction) to 1219.669 (after weir
construction). But 1219.144 is higher than the existing left bank level which has RL of 1219.279. Therefore the difference 1219.144 - 1219.279 plus a free board of 0.5m bund should be constructed at both banks on the U/S.

\[(1219.669 - 1219.279) + 0.5 = 0.89\text{m}\]

The construction soil is available in the bank vicinity

6.2 Changing river course

At about 480mt length from the weir site the river is approaching closely to the canal and for some distance the right bank of the river is the same as the left bank of the canal. Slight erosion at the right bank of the river will eat a way the canal bank. Therefore at this point the river course should be changed far away from the canal bank and the bank should be left protected for the natural vegetation to grow.

7. Infrastructures in the scheme

7.1. Weir (the design plan is annexed)

Weir dimensions

Bottom width \((B)\)

\[
B = \frac{H_e + \text{weir height}}{\sqrt{G-1}}
\]

Let \(G\) for gabion be 90% stone (assuming 10% void space)

\[
G_{\text{stone}} = 2.65
\]

\[
G_{\text{gabion}} = 2.385
\]

\[
= 1.01 + 0.504
\]

\[
= \sqrt{2.385 - 1}
\]

\[
1.514 = 1.286\text{m} \quad \text{say -1m}
\]
Top width ($B'$)

$$B' = \frac{H_e}{\sqrt{G-1}} = 1.01 = 0.858 \text{ say } 1 \text{m}$$

Creep length ($L$)

$$L = C \cdot H_L$$

- $C$ for coarse sand bed = 12
- $H_L = \text{difference in U/s and d/s WL}$
  - $H_L = 19.669 - 12.02 = 0.649$
  - Adding 0.5 for d/s river bed retrogression
  - $H_L = 0.5 + 0.649 = 1.149$
  - $L = 12 \times 1.149 = 13.788 \text{m}$

D/s apron length ($L_3$)

$$L_3 = 5 \ (Y_2 - Y_1)$$

$$= 5 \ (1.154 - 0.351)$$

$$= 4.015 \text{ take } 4 \text{m}$$

Thickness of floor ($t$)

$$t = \frac{Y_o}{L} = 1.848 \text{ take } 0.134 \text{ take } 0.5$$

7.2 Spill way

Provision of spill way is an important requirement for the rejection of sediment loads and control of excessive flows into the canal.

From the top map the RL of the spill way bed is 1217.63 and the average canal bed is chosen to be 1218.02 to give adequate gradient towards the spill way.

(The design and dimensions are annexed)
7.3 Outlets

The 1st command at the left of the canal is located about 480 m from the weir axis. The RL of this plot is much lower than the canal bed and FSL of the canal. In trying to cut an outlet to water that piece of land (about ½ Ha), the main canal is severely damaged and is difficult to rehabilitate so that water could pass to the main command in the lower reaches. Therefore a concrete tube outlet of adequate size should be provided for such kinds of outlet sets.

\[ Q = CA \sqrt{2yH_o} \]
\[ C = 0.65 \]
\[ H_o \text{ free head water from the centre of the ring to full supply level of the canal (0.8m)} \]
\[ A = \text{X area of the ring} \]
\[ d = \text{tube diameter} \]
\[ = \frac{0.65 \times \pi(0.2)^2}{4} \sqrt{2 \times (9.81)(0.8)} \]
\[ = 0.0798 \text{m}^3/\text{sec} \]

However, as described below 500m$^3$/ha/application is assumed for the design

\[ > 250 \text{ m}^3/0.5\text{ha/application} \]

\[ > \text{In four hours of flood, } 0.0798 \times 4 \times 60 \times 60 \text{ sec } = 1149.12 \text{m}^3 \]

1149.12 > 250\text{m}^3 \Rightarrow \text{capacity of the pipe is enough}

Alternatively, 1hr of flood can water the 0.5ha plot through 0.2m diameter tube outlet.

\[ => \text{8. Crop water requirement and canal capacity} \]

The major crops grown in the command area are Maize and Sorghum. The altitude of the project area is about 1220 m.a.s.l and a mean annual temp of 22°C.
In such climatic regions in general, for 5 months growing period of maize, it requires a total of 500 – 600 mm of water. And sorghum requires 450 – 650 mm of water in 7 months growing period.

By local enquiring of beneficiaries, the average and frequent flow in the river stays for about 4 hrs. and the return period of such amount of flood during the 7 month crop growing periods is 15 times.

- Gross quantity of flow in the river will be

\[
4 \text{ hr} \times 15 \text{ times} = 60 \text{ hr of flow} \\
60 \times 60 \times 60 = 216,000 \text{ sec.}
\]

Canal capacity

The existing canal has the following average canal dimensions

- Canal bed width = 1.76m
- Average to width = 4.33m
- Average depth = 2.356m (say 2 effective)
- General slope = 0.008
  \[n = 0.028\]

Capacity

\[Q = V.A.\]

\[V = \frac{1}{n} R^{0.66} S^{0.55} \sqrt{\frac{R}{A}}\]

\[R = \frac{A}{P}\]

\[X = \text{area} = \frac{1}{2} (4.33 + 1.76) \times 2 = 6.69\]
Perimeter = $2(3) + 1.76 = 5.76m$

\[ R = \frac{6.09}{5.76} = 1.06 \]

\[ V = \frac{1}{0.028} \times (1.06)^{0.6} \times (0.008)^{0.5} \]

\[ = 3.32 \text{ m/s} \]

1.263 m/s (erosive velocity), implies that a drop structure is essential. However, the attained gradient is due to broken canal to water a low spotted plot from the main canal. This outlet will be changed to pipe outlet so that the original bed slop will be regained.

\[ Q = V.A \]

\[ 3.32 \times 6.09 = 20.2 m^3/s \]

This implies that the existing canal capacity is much more than the required dimension. Therefore, the quantity of flow should be controlled at the approaching canal using the sluice gate.

The dimension of the approach canal is determined earlier.

Width = 2m
Depth = 0.5m
Slop = 0.0137
\[ P = 2(0.5) + 2 = 3m \]
\[ A = 2 \times 0.5 = 1m^2 \]
\[ n = 0.028 \]
\[ R = \frac{1}{n} = 0.33 \]

\[ V = \frac{1}{0.028} \times (0.33)^{0.6} \times (0.0137)^{0.5} = 2.01 m^3/s \]

\[ Q = V.A = 2.01 \times 1 = 2.01 m^3/s \]

Assuming that 25% losses due to friction and bed siltation water that will flow to the canal.
2.01 m³/s x 216,000 sec = 434,160 m³ of water will get in to the canal through out the growing period of the crop.

But maize needs 500mm of water on average through out the growing period and if an irrigation interval of 15 days in assumed for maize.

Number of irrigations required will be $\frac{20 \times 5}{15} = 15 = 10$ times

Amount of water per application $= \frac{500}{10} = 50$ mm

$= 500$ m³/ha

As discussed above the majority of the command area need 3 applications for the crops to mature, therefore $3 \times 500$ m³/s of water is needed for a hectare of plot.

Therefore the total amount of land that would be irrigated by the canal will be

$\frac{434,160}{1500} = 289.44$ ha

(Assuming 40% water losses in conveyance applications)

a total of 173 ha of land can be cultivated.
### B.O and project cost

#### 8.1 Cash

<table>
<thead>
<tr>
<th></th>
<th>Quantity</th>
<th>Unit Cost</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Gabion wire</td>
<td>205.8 m³</td>
<td>280.00</td>
<td>57,624</td>
</tr>
<tr>
<td>B. 20cm concrete ring</td>
<td>20 m</td>
<td>70</td>
<td>1,400</td>
</tr>
<tr>
<td>C. Stone transportation (25 dump)</td>
<td>25 dump</td>
<td>180</td>
<td>4,500</td>
</tr>
<tr>
<td>D. Filter fabric (13x22) m²</td>
<td>286 m²</td>
<td>40</td>
<td>11,440</td>
</tr>
<tr>
<td>E. Sluice gate (LS)</td>
<td>2 Pe</td>
<td>2,000</td>
<td>4,000</td>
</tr>
<tr>
<td>F. Masonry cost</td>
<td>11.8 m³</td>
<td>500</td>
<td>5,900</td>
</tr>
<tr>
<td>G. Supervision cost (Forman)</td>
<td>1 month</td>
<td>400</td>
<td>400</td>
</tr>
<tr>
<td>H. Surveying level (sokkia)</td>
<td>With accessories</td>
<td>1</td>
<td>1,000</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>300.00</td>
<td>600</td>
</tr>
</tbody>
</table>

**Total cost** = 95,824

10 % contingency = 9,582.4

Total cost = 105,406.4

### 8.2 Grain

#### A. Stone collection (205.8 m³)

Assuming that 0.3 m³/MD

\[
\begin{align*}
205.8 \text{ MD} &= 686 \text{ MD} \\
0.3 &= 686 \times 3 \text{ kg} = 2058 \text{ kg}
\end{align*}
\]
B. Loading stone (205.8m$^3$)

Assuming that 10 people can load 1 dump truck of stone/day.

- $8\text{m}^3/10 \text{ MD}$
- $0.8\text{m}^3/\text{MD}$

$\Rightarrow 205.8\text{m}^3 \text{ MD} = 257.25 \text{ MD}$

$\Rightarrow 257.25 \times 3 \text{ kg} = 771.75 \text{ kg}$

C. Excavations (205.8)

Assuming that $1\text{m}^3 / \text{MD}$

Total MD required $205.8\text{m}^3 \div 1\text{m}^3 = 205.8 \text{ MD}$

Total grain required $205.8 \times 3 = 617.4 \text{ kg}$

D. Marginal bunds

- X - area $= \frac{1}{2} (1+6.34) 0.89 = 3.266 \text{m}^3$
- Volume per linear metre $= 3.22 \text{ m}^3/\text{m}$

Total length of the bund

$= 10 + 30 = 40\text{m} \text{ (from top map)}$

Total volume of earthwork (bund construction)

$= 3.266 \times 40 \text{ m}^3 = 130.652 \text{ m}^3$

Assuming $0.20\text{m}^3/\text{MD}$

- Total MD required $130.652 \div 0.20 = 653.26 \text{ MD}$

- Total grain required $= 653.26 \times 3 \text{ Kg} = 1959.78 \text{ Kg}$
E. Filling of gabions with stone

Assuming 0.3m$^3$/MD

$$\text{Total MD required} = \frac{205.3 \text{ MD}}{0.3} = 684.3 \text{ MD}$$

Total grain required = 684.3 kg x 3Kg = 2053Kg

F. Changing River direction

- Depth = 2.5 m
- Width = 6m
- Length = 60m
- Volume = 60x6x2.5m = 900m$^3$

Assuming 0.5m$^3$/MD

$$\text{Total MD required} = \frac{900 \text{ m}^3}{0.5 \text{ m}^3} = 1800 \text{ MD}$$

Total grain required = 1800 x 3Kg = 5400Kg

Gross total grain required for the whole activity

$$2058 + 771.75 + 147 + 617.4 + 1959.78 + 2053 + 5400 = 12859.93 \text{ Kg}.$$
10. Operation Maintenances

The catchment area of the water source of the diversion in scarcely covered with vegetation. Because of high erosion in the catchments, there is a possibility of silt deposit in front of the weir and in the approach canal. These deposits of silt will definitely block the flow of water through the canal. Therefore, clearance of the silt deposit & trash from the canal must be done regularly & frequently. This task is not difficult for the Konso people as they have the skills & patience to rehabilitate frequently demolished traditional diversion spurs. Besides, in times where water is not necessary for irrigation, the gate of sluiceway at the spillway should be kept open while the gate at the canal head regulator is fully closed. This will flush the silt deposit in the approach canal & at the back of the weir.

In times when there is excess flood in the stream it has to be checked from entering to the cultivable land by closing the entrance gate on the canal head regulator. If the excess water passes into the fields it can cause heavy damages to the canal and farmlands.

To operate the gates at the spillway a person who resides near the headwork should be appointed and he should be present when there are flood flows in the stream.